Appendix B

Water Resources Evaluation of Amargosa Creek

Prepared for the City of Palmdale

July 2009

From Science to Solutions

TABLE OF CONTENTS

LIST OF FIGURES

LIST OF TABLES

EXECUTIVE SUMMARY

The City of Palmdale proposes to develop the Upper Amargosa Project (UAP) within the City limits to increase the groundwater recharge capabilities and groundwater supplies in the Antelope Valley by diverting a portion of the streamflow in Amargosa Creek. Amargosa Creek is an ephemeral stream that is dry most of the time but also characterized by short-duration, high-flow events that exceed the percolation capacity of the channel. The water produced by these high-flow events travels downstream to an area of the channel underlain by clay deposits that limit recharge, thereby causing the excess streamflow to pond in old lakebeds and subsequently evaporate back to the atmosphere. The UAP would include the following components:

1) Approximately 20-acres of ponds for recharge, both off-channel and in-channel and associated infrastructure;
2) A community nature park of nearly 40 acres consisting of multi-use pathways, picnic tables, interpretive plaques, and habitat enhancement/restoration areas;
3) A native habitat conservation area of roughly 20 acres; and
4) Existing open stream channel.

The recharge facility would receive water from two sources, the State Water Project (SWP) and the Amargosa Creek watershed. The total combined (SWP water and Amargosa Creek stormwater runoff) annual average available water for the UAP would be approximately 25,400 AFY.

The portion of the project discussed herein is the diversion of streamflow from Amargosa Creek to the UAP recharge facilities under a diversion permit issued by the State Water Resources Control Board (SWRCB) pursuant to an application filed by the City to divert a portion of the streamflows occurring in Amargosa Creek at the proposed Point of Diversion (POD). Two key requirements of the SWRCB Application are that the applicant states the requested rate of diversion and the maximum annual diversion amount. Listed below are the amounts set forth in the application:

1. Maximum Rate of Diversion = 100 cubic feet per second (cfs); and,
2. Maximum Annual Diversion Amount = 2,700 acre-feet.

The maximum rate and amount diverted shown above and set forth in the application does not mean that the UAP will be operated in a manner that will preclude channel recharge downstream of the POD. The SWRCB will review the application and approve or disapprove based on the merits. If the application is approved, the SWRCB will issue a permit that will define the operating conditions of the UAP. An example of an operating condition that may be incorporated in the permit is to not divert that portion of the natural flow above the UAP PODs that would percolate in the stream channel downstream.

PRINCIPAL FINDINGS

1. The concept of the UAP is to divert the portion of the streamflow in Amargosa Creek that is evaporated in either Lake Lancaster at Avenue H, Piute Ponds or Rosamond Dry Lake and is lost to beneficial use.
2. The estimated recharge capacity of the proposed ponds is 100 AF per day, or the equivalent of about 50 cubic-feet-per second. The SWRCB Application used a diversion rate of 100 cfs because the discharge from Amargosa Creek watershed will likely occur over periods of hours, rather than days.

3. Channel bed seepage occurs along the length of the Amargosa Creek down-stream from the UAP for approximately ten miles to north of Avenue J where finer silt and clay playa deposits impede seepage and recharge to the principal aquifer.
4. Amargosa Creek is tributary to Lake Lancaster (detention basin north of Avenue H), Piute Ponds, and then Rosamond Dry Lake. The Amargosa Creek watershed upstream of the POD is 29 square miles, which is approximately 20 percent of the watershed of Lake Lancaster (160 square miles) and approximately 2 percent of the watershed area of Rosamond Dry Lake (1,200 square miles).
5. For the Amargosa Creek watershed, daily rainfall on average exceeds 1 inch on six days each year in the mountains and 2 days each year in the valley. In the mountains rainfall is expected to exceed 0.2 inches each hour 23 hours each year and 0.5 inches per hour 2 hours each year.
6. A daily runoff model of Amargosa Creek was developed using rainfall records and nearby gaging stations records from Little Rock Creek because there is no historical gaging station on the mainstem of Amargosa Creek.
7. The average annual Amargosa Creek streamflow at the POD is estimated to be 2,600 AFY (section 4.2.2). Downstream of POD to Avenue J, urban runoff contributes an estimated 1,100 AFY on average to Amargosa Creek streamflow (section 4.3.2). Of the combined flows (3,700 AFY), 2,200 AFY is estimated to seep into the channel bed between the POD and Avenue J and provides recharge to the aquifer (section 4.3.3), and 1,500 AFY is estimated to flow past Avenue J and eventually flow into Lake Lancaster at Avenue H, Piute Ponds or Rosamond Dry Lake where recharge is limited due to the finer sediments of the historical and existing lakebeds (section 4.3.4).
8. The diversion potential, which is the maximum diversion that is possible from the streamflow at the POD, is 1,100 AFY on average (section 4.2.5). The diversion at POD based on streamflow at Avenue J is the volume that could be diverted without reducing the existing channel seepage between the POD and Avenue J. and is estimated to be 400 AFY (section 4.3.5). Total runoff at Avenue J after the proposed diversion is 1,100 AF on average.

Table ES-1: Amargosa Creek Streamflow Summary

Year		Volumes	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Water Year
Average	Current	Streamflow at POD	446	543	293	221	17	0	3	0	17	0	505	655	**2,616**
		Urban runoff POD to Ave J	213	189	110	90	26	0	2	0	30	13	265	211	**1,116**
		Channel Seepage POD to Ave J	352	403	355	281	34	0	5	0	34	10	412	396	**2,227**
		Total Streamflow at Ave J	307	329	48	30	10	0	0	0	13	3	359	470	**1,506**
	Proposed	Diversion Potential at POD	135	208	252	193	17	0	3	0	15	0	162	185	**1,147**
		Diversion based on Streamflow at Ave J	70	86	39	29	5	0	0	0	2	0	104	83	**405**
		Streamflow after Diversion at Ave J	237	243	9	2	4	0	0	0	11	3	256	387	**1,101**
Maximum (WY 1969)	Current	Streamflow at POD	4,979	3,877	347	736	0	0	40	0	0	0	64	0	**10,004**
		Urban runoff POD to Ave J	687	878	11	137	0	0	27	0	0	0	128	0	**1,847**
		Channel Seepage POD to Ave J	2,343	2,107	347	758	0	0	66	0	0	0	163	0	**5,734**
		Total Streamflow at Ave J	3,323	2,649	10	114	0	0	2	0	0	0	30	0	**6,117**
	Proposed	Diversion Potential at POD	930	987	284	495	0	0	40	0	0	0	64	0	**2,762**
		Diversion based on Streamflow at Ave J	706	593	10	114	0	0	2	0	0	0	30	0	**1,433**
		Streamflow after Diversion at Ave J	2,617	2,056	0	0	0	0	0	0	0	0	0	0	**4,684**
Cumulative	Current	Streamflow at POD	5,799	7,054	3,807	2,871	224	0	40	0	221	4	6,066	7,925	**34,010**
		Urban runoff POD to Ave J	2,769	2,459	1,431	1,170	340	0	27	0	395	158	3,186	2,576	**14,511**
		Channel Seepage POD to Ave J	4,581	5,242	4,613	3,647	436	0	66	0	443	120	4,941	4,863	**28,950**
		Total Streamflow at Ave J	3,988	4,271	625	394	128	0	2	0	173	42	4,310	5,638	**19,572**
	Proposed	Diversion Potential at POD	1,751	2,700	3,270	2,510	224	0	40	0	194	4	1,938	2,284	**14,913**
		Diversion based on Streamflow at Ave J	914	1,114	510	374	70	0	2	0	32	3	1,243	996	**5,259**
		Streamflow after Diversion at Ave J	3,075	3,156	115	20	58	0	0	0	141	39	3,067	4,642	**14,313**

9. The effect the diversion would have on the seasonally flooded areas downstream of Lake Lancaster and the seasonal flooding of Rosamond Dry Lake is minimal. The reduction in volume of seasonal flooding at Rosamond Dry Lake due to the diversion at the POD is approximately 1 percent.
10. There are no water quality issues of concern.
11. Recharge operations at the UAP will augment groundwater supplies to the north and east.

1 INTRODUCTION

The physical setting of the proposed Upper Amargosa Project (UAP), and the geology and hydrology of the Amargosa Creek watershed are presented in the following sections, including the fundamental data and analyses supporting the application to the State Water Resources Control Board (SWRCB) for the right to divert streamflow, and the potential environmental impacts to the geology and hydrology resulting from the proposed diversion of a portion of the streamflow to the UAP.

1.1 PROJECT

The City proposes to develop the UAP on approximately 87 acres located within the City limits (Figures 1-1 and 1-2). The UAP would include the following components:

5) Approximately 20-acre recharge facility, including off-channel and in-channel recharge basins and infrastructure;
6) a 38-acre community nature park containing multi-use pathways, picnic tables, interpretive plaques, and habitat enhancement/restoration areas;
7) a 22-acre native habitat conservation area; and
8) 7 acres of open stream channel.

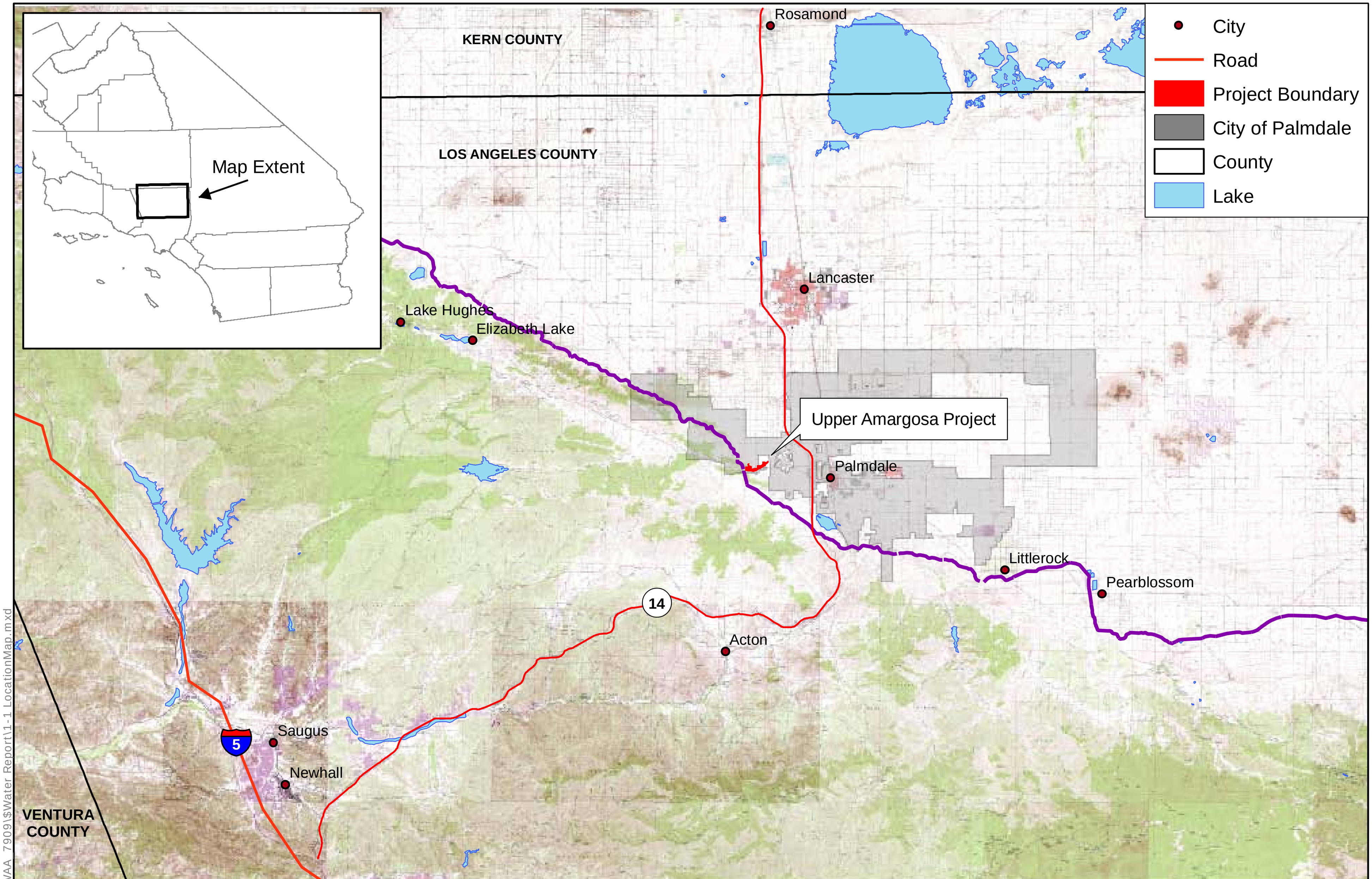
The purpose of this recharge facility would be to provide increased groundwater recharge to the Antelope Valley Groundwater Basin. The recharge facility would receive water from two sources, the State Water Project (SWP) and the Amargosa Creek watershed. The recharge facility would consist of two in-channel basins and six off-channel basins designed to retain water and allow it to infiltrate into the ground. Maximum recharge estimates, based on a full year operation schedule range from approximately 22,000 acre feet per year (AFY) to 80,000 AFY, and would average 36,500 AFY. Based on the proposed operation schedule where recharge basins would be out of operation during summer months when water may not be available, the recharge facilities would recharge between 14,500 AFY to 53,000 AFY, and would average approximately 24,300 AFY. The total combined (SWP water and Amargosa Creek stormwater runoff) annual average available water for the UAP would be approximately 25,400 AFY.

The three local state water project contractors (Antelope Valley-East Kern Water Agency [AVEK], Palmdale Water District [PWD], and Littlerock Creek Irrigation District [LCID]) would, following negotiation of a Memoranda of Understanding (MOU), deliver a portion of their available SWP water supply to the UAP recharge facility. The project, under the planned MOU, would divert an average of approximately 24,300 AFY of their currently unused SWP allocations for recharge (Kennedy/Jenks 2008). There is also a potential to obtain additional water from other SWP contractors when their SWP allocations exceed existing water demands. The project would also divert stormwater from Amargosa Creek to the UAP recharge facilities under a diversion permit that would be obtained from the State Water Resources Control Board (SWRCB) pursuant to an application to divert stream flow. Water diverted to recharge the Antelope Valley Groundwater Basin aquifer could then be extracted from the basin at a later date for use within the City and surrounding communities.

In conjunction with the recharge facility, a community nature park would be created within the boundaries of the project site. The nature park would provide recreational and educational opportunities, including 2.5 miles of multi-use pathways through the nature park and around the proposed recharge basins. The pathways would facilitate the community’s continued use of the area and link to existing trails and bike pathways within the City. Passive recreational amenities (i.e., ramadas and picnic tables) would be placed within the park. The nature park would include the enhancement and restoration of previously disturbed habitat to remove non-native vegetation and restore native Mojave Desert scrub, riparian vegetation, and wildlife habitat. Educational displays and interpretive plaques would be located throughout the nature park to provide information on local biological and water resources (i.e., desert environment, native plants and animals, watershed processes, urban runoff, and the recharge facilities).

Twenty-two (22) acres of upland area in the northwestern portion of the project site would be dedicated as a Native Habitat Conservation Area. This area consists of mostly undisturbed habitat (i.e., low shrubs, cacti, mature juniper and annual wildflowers and would be preserved in perpetuity.

The project would also include 7 acres of open stream channel.



File: T:\WTR_Palmdale\WAA_79091$Water Report\1-1 LocationMap.mxd

NOTES:

Coordinate System: CA State Plane Zone 5 (feet)
Horizontal Datum: NAD 83

Upper Amargosa Project Site

0 2 4 8 12 16

Miles

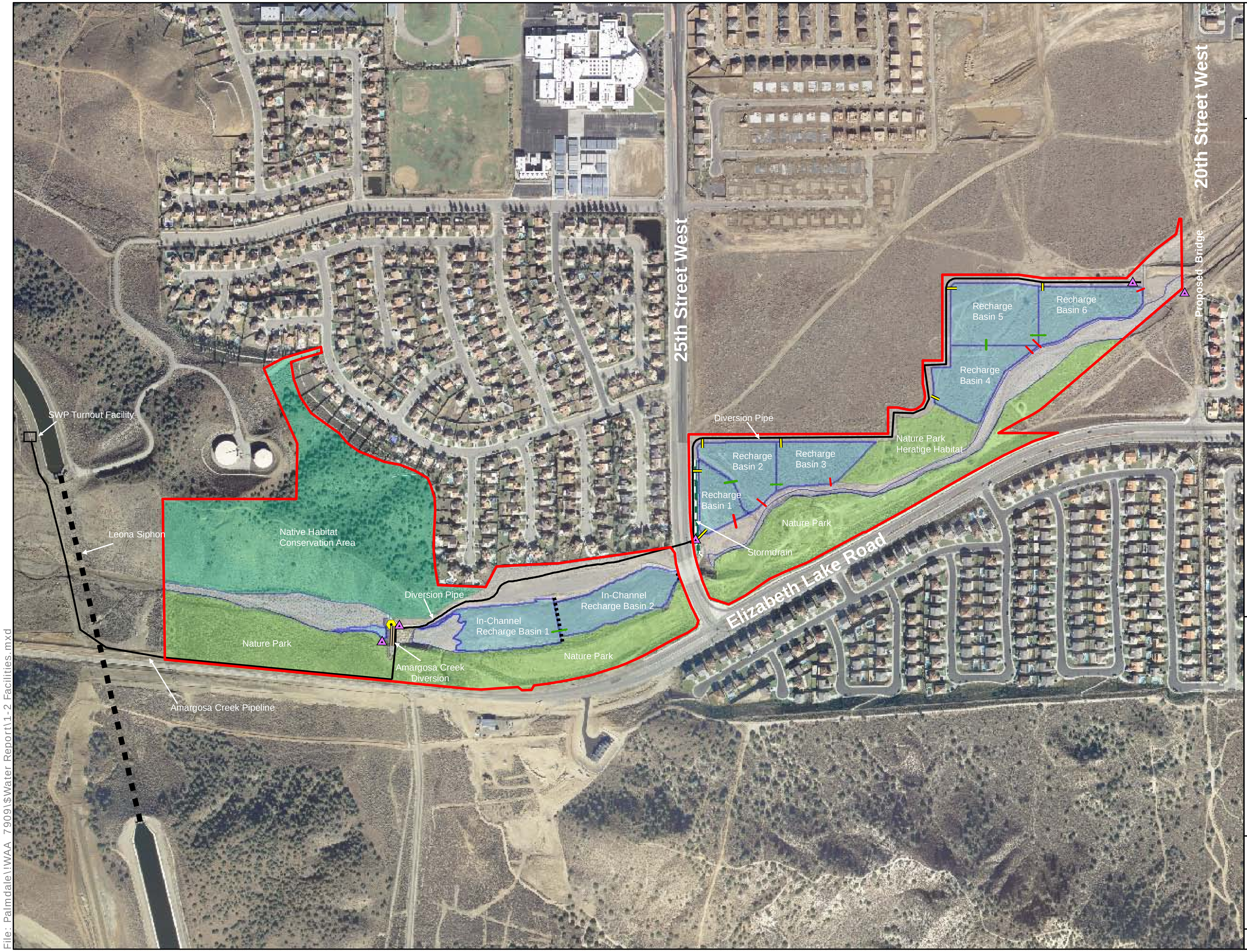
SAIC
From Science to Solutions

DATE: 06/10/08

BY: D Beckwith

FIGURE:

1-1



Upper Amargosa Project

- Groundwater Recharge Facilities -

- Project Boundary (87 acres)
- Recharge Basin (20 acres)
- Nature Park (28 acres)
- Native Hab. Conserv. (22 acres)
- Open Stream Channel (7 acres)
- Unclassified (10 acres)
- Push-Up Dam
- Conveyance Pipe
- Storm Drain with Outlet Structure
- Basin Inlet Pipe/Valve with Flow Measurement
- Interpond Pipe/Gate/Weir with Flow Measurement
- Return Flow Pipe/Gate/Weir with Outlet Protection/Flow Measurement
- Stream Diversion Intake Structure/Headgate
- Flow Measurement Device

0 250 500 750 1,000

Feet

N

NOTES:

Coord Sys: State Plane NAD 83 Zone 5 U.S. Foot
Basemap: LARIAC 4-in resolution Airphoto, 2006

SAIC
From Science to Solutions

FIGURE:

1-2

DATE: 06/27/08 BY: A Pappas

File: Palmdale\IWAA 7909\SWater Report\1-2 Facilities.mxd

2 GEOLOGY

Antelope Valley Groundwater Basin is a large sediment-filled structural depression that is a down-faulted block between the San Andreas and the Garlock faults. The basin is filled with unconsolidated alluvium and lacustrine deposits. The fine-grained lacustrine deposits accumulated in a large lake or marsh that at times covered the area. Alluvial fans that formed by the deposition of eroding materials from the up-faulted block of the Sierra Pelona and San Gabriel Mountains encroached upon the ancient lake where the lacustrine deposits were accumulating, forcing the ancient lake and associated lacustrine deposits to the north and to its present location at Rosamond Dry Lake and Rogers Dry Lake (USGS 2003). These lacustrine deposits are overlain by as much as 800 ft of alluvium in the southern part of the Lancaster subunit near Palmdale and become progressively shallower northward, being exposed at the surface near the southern edge of Rosamond Dry Lake and Rogers Dry Lake. Antelope Valley is a hydrologically-closed basin, whereby water leaves the basin only by evaporation. The Amargosa Creek watershed is defined by the area from which the Amargosa Creek collects and concentrates runoff from rainfall, and it overlies the transition between the up-faulted block of the Sierra Pelona Mountains and the down-faulted block of the Antelope Valley (Figure 2-1).

2.1 DATA SOURCES

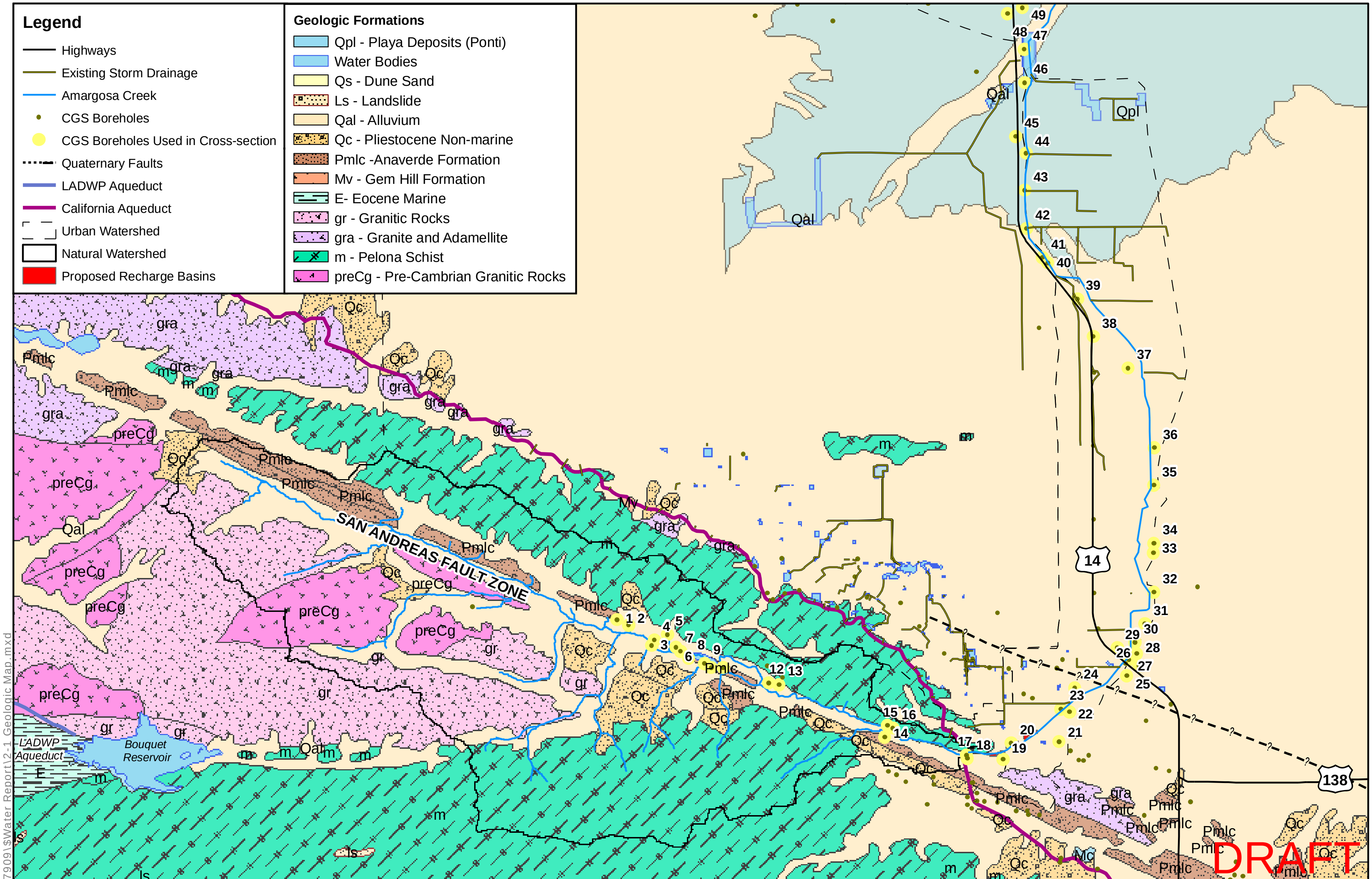
The California Department of Water Resources (DWR) collaborated with the United States Geologic Survey (USGS) in the 1960s to summarize the well data for the Antelope Valley and published the information in Bulletin 91 (DWR 1962 and DWR 1966). The area from the UAP to Rosamond Dry Lake is covered by Bulletins 91-6 and 91-12. Additional well log data was requested from the DWR by the Technical Committee for the Antelope Valley Groundwater Adjudication Area. These additional well logs were provided to SAIC by Luhdorff & Scalmanini within Township and Range 6N-12-13W and 7N12W (Sections 25 to 36). Well log location and elevation was determined from Bulletin 91, USGS groundwater database, and from the descriptions of the well location in the well log. For each well log, the ground surface, top of the clay layers, bottom of the clay layers, and bedrock were plotted in the profile if it was available.

The Geologic Map of California: Los Angeles Sheet (Jennings 1969) was digitized for the Antelope Valley Groundwater Litigation and provided to SAIC (Figure 2-1). The playa lake bed deposits (Qpl) were mapped by Ponti and others (1981). The Qpl boundary was digitized from the California Geologic Survey seismic hazard evaluations for Lancaster West, Lancaster East, and Rosamond quadrangles.

California Geologic Survey compiled borehole data for the development of their Seismic Hazard evaluations (CGS 2002, 2005). Borehole data is available for Sleepy Valley, Ritter Ranch, Lancaster West, and Rosamond 7.5 USGS quadrangles and covers most the Amargosa Creek drainage. Boreholes are generally shallow and rarely exceed 100 ft in depth. The local borehole data utilized in this evaluation is plotted in Figure 2-2. The lithologic data for boreholes near the Amargosa Creek are represented in Figure 2-3. The ASTM based lithologic codes for each logged interval were simplified to three classes based on the first letter (S=Sand=, M=Silt, and C=Clay).

2.2 STRUCTURAL GEOLOGY

The dominant structural feature is the north-west to south-east trending San Andreas Rift Zone which is the boundary between the Pacific and North American Plates. The San Andreas Fault movement has created the linear, trough-like valley known as the Leona Valley which is part of the Amargosa Creek Natural Watershed. The north-east to south-west trending Garlock Fault is truncated at the San Andreas Rift Zone and is the northern boundary to the Antelope Valley Groundwater Basin.



NOTES:
Coordinate System: Albers Conical Equal Area
Horizontal Datum: NAD 83
Source: Jennings, C.W. and Strand, R.G. 1969
Geologic Map of California:
Los Angeles Quadrangle
Qpl - Ponti and others 1981

Geologic Map of the Amargosa Creek Watershed

0 0.5 1 2 3 4
Miles

SAIC®
From Science to Solutions

FIGURE:
2-1

DATE: 10-02-08
BY: JD

File: Palmdale\TB_WAA_7909\$Water Report\2-1 Geologic Map.mxd

The USGS recognizes a potential un-named buried fault that runs parallel to the mountain front approximately 2 miles to the northeast from where Amargosa Creek enters the Antelope Valley at the 25th Street Bridge (Figure 2-1) (Christensen 2004). This un-named fault could potentially be a barrier to groundwater flow. It is unlikely that the unnamed fault is a significant barrier to groundwater flow, because there is no evidence of water emerging at the ground surface in the form of springs and wetlands between the mountain front and the fault.

2.3 MOUNTAIN BLOCK

The Amargosa Creek watershed above the UAP (Natural Watershed) is an asymmetric palmate-shaped drainage network incised into bedrock of the Sierra Pelona Mountains where a thin veneer of coarse grained alluvium is deposited along the drainage network. The Natural Watershed overlies the San Andreas Fault Zone and has been offset from its alluvial fan by the right-lateral strike-slip movement between the North America and Pacific tectonic plates.

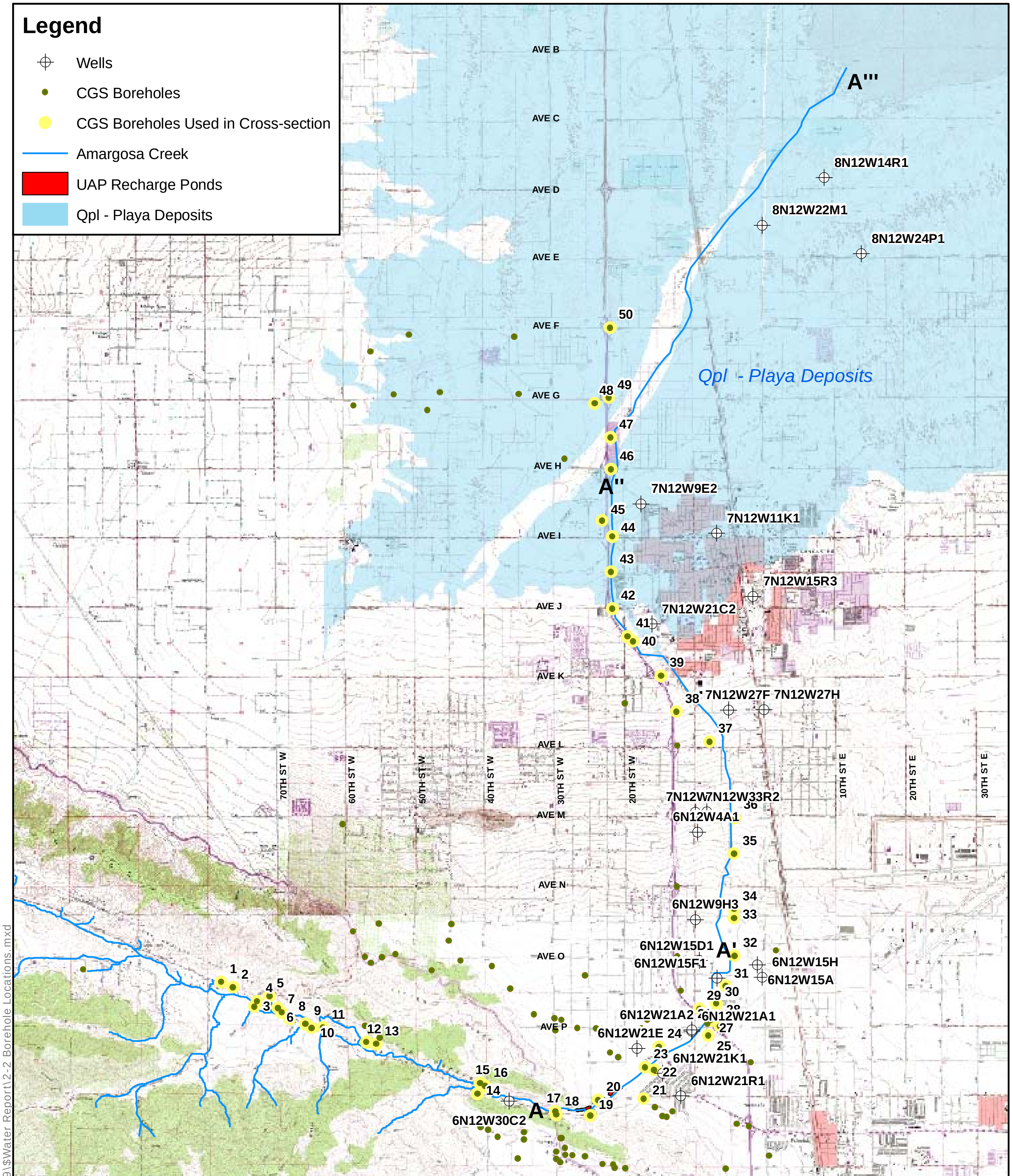
Bedrock within the Natural Watershed generally consists of Pre-Cambrian to Eocene granitic and metaphoric rocks including the Pelona Schist Formation. The younger Anaverde Formation occurs as elongated outcrops associated with the San Andreas Fault Zone, and recent Quaternary alluvial materials have been deposited where streamflow has incised depressions into bedrock.

The Pelona Schist Formation is comprised of highly deformed and metamorphosed sedimentary rocks which occur mainly along the San Andreas and Garlock Fault Zones. The granitic rocks were emplaced as large plutons (intrusive magma bodies). The Anaverde Formation is comprised of Pliocene non-marine fluvial sandstones, lacustrine clays, and thin beds of gypsum evaporate, that have been highly deformed by faulting along the San Andreas Fault Zone. Quaternary alluvium is generally comprised of coarse to medium grain granite grus and eroded schist, with minor amounts of silt and clay. Based on the borehole data, the alluvium in the Natural Watershed is shallow with the bedrock ranging from 8 to 70 feet below the ground surface (ft bgs).

2.4 VALLEY BLOCK

The Amargosa Creek watershed downstream of the UAP (Urban Watershed) occurs on part of an alluvial fan in the Antelope Valley where the City of Palmdale and the City of Lancaster have developed the landscape and altered the natural drainage with engineered systems to convey stormwater from the urban area to the Amargosa Creek and ultimately to the Rosamond Lake. The alluvial fan head originates from where the Amargosa Creek crosses from the up-faulted block of the mountain front to the down-faulted block under the Antelope Valley (Figure 2-1). In the Antelope Valley, deposits of medium to coarse grain material overlie fine grain lacustrine deposits and the playa lake bed deposits (Ponti et. al., 1981). Additionally, borehole and well data have been compiled to corroborate the occurrence of lithologic faces indicative of the lacustrine environment (Figure 2-3). Borings were advanced to generally between 30 feet and 70 ft bgs across the Antelope Valley, and logs prepared show that playa lake bed deposits occur near surface from Avenue J to the north including Rosamond Dry Lake (Figure 2-3).

Wells have been advanced to between 300 feet to 1000 ft bgs across the Antelope Valley, and the logs prepared show ancient lake bed lacustrine deposits occur at approximately 800 feet depth from 10th Street to Avenue J (Figure 2-2). The coarse grain sediments overlying the lacustrine deposits comprise the unconfined “principal aquifer”, and the confined coarse grain sediments below the lacustrine deposits comprise the “deep aquifer”.



NOTES:
Coordinate System: UTM Zone 11N
Horizontal Datum: NAD 83
Topo Map: USGS 24K
Qpl - Ponti and Others

Amargosa Creek
Borehole and Well Log Locations

0 0.5 1 2 3 4
Miles

SAIC®
From Science to Solutions

FIGURE:
2-2

DATE: 10-27-08 | BY: JD

File: T:\Palmdale\WAA_7909\$Water Report\2-2 Borehole Locations.mxd

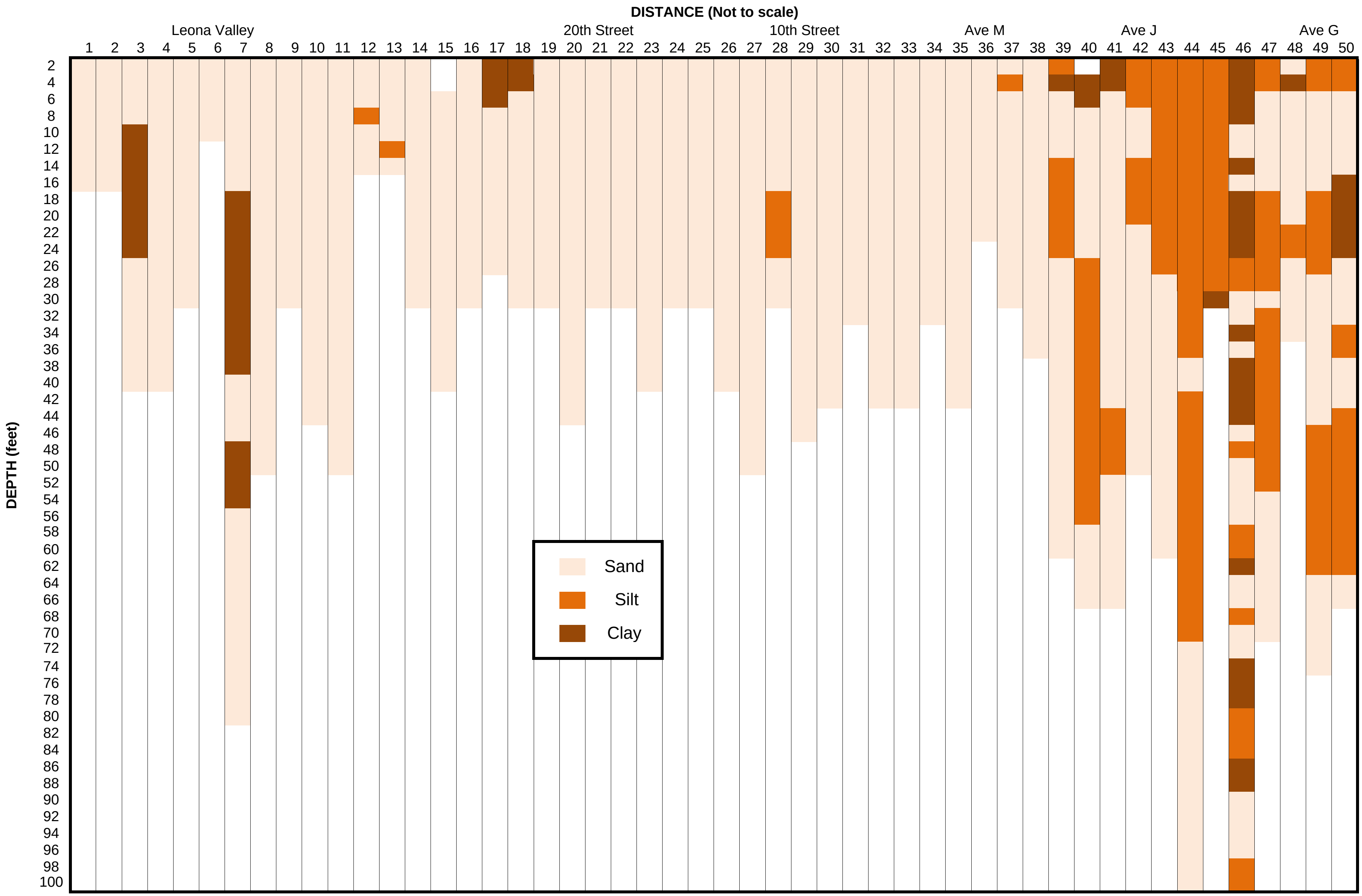


Figure 2-3: California Geologic Survey Compiled Shallow Boring Logs

2.5 NATURAL WATERSHED CHARACTERISTICS

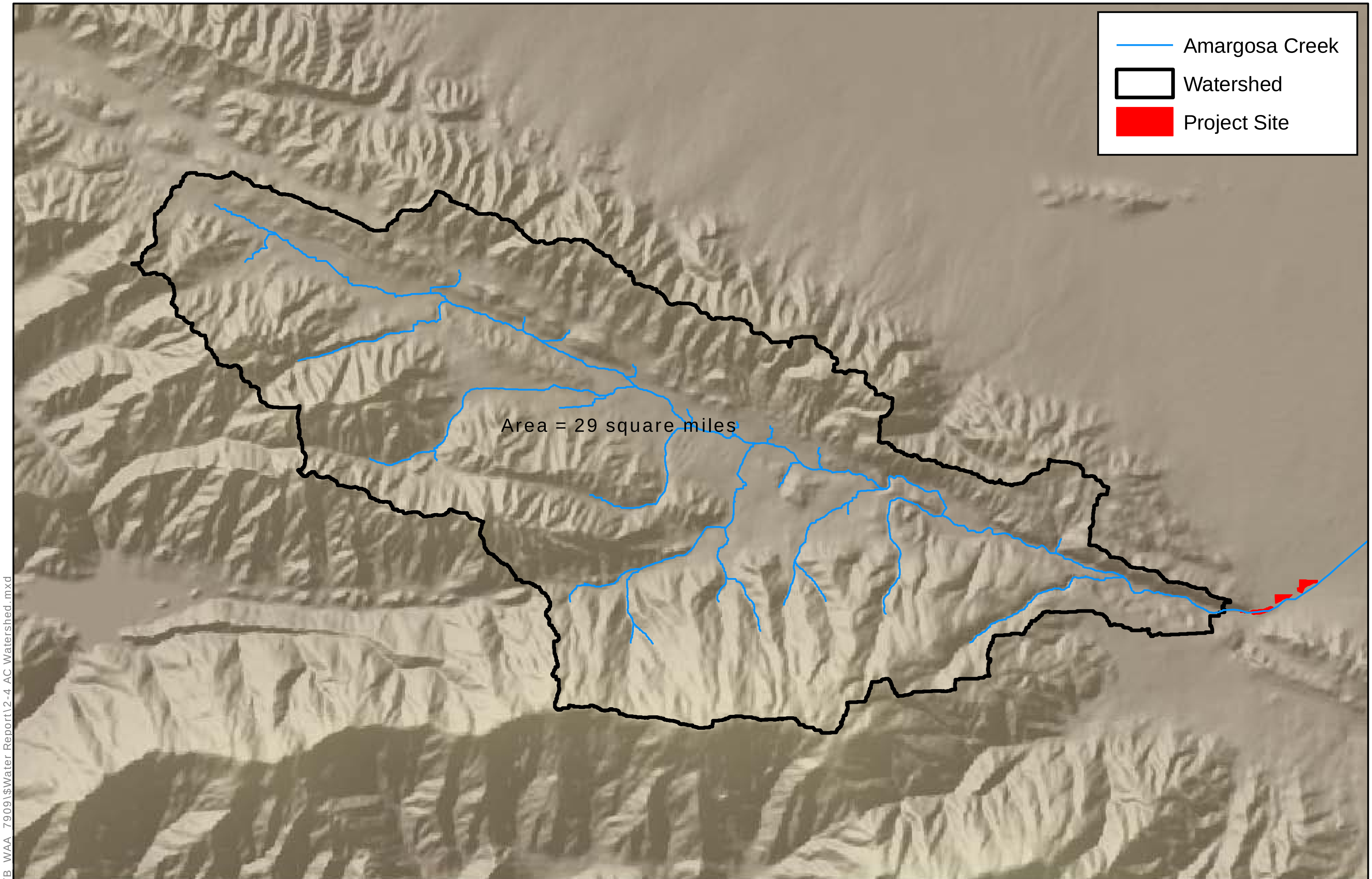
The Natural Watershed is defined as the topographic area that contributes surface runoff to the proposed upstream point of diversion (POD), located in western portion of the project area. A 10-meter digital elevation model (DEM) representing the topography within the Natural Watershed and surrounding area was obtained from the National Elevation Dataset (USGS 2008). The watershed was delineated from the DEM with HEC-GeoHMS, a GIS software developed by the United States Army Corps of Engineers. The watershed boundary was checked against USGS topographic map of the watershed (Figure 2-4).

The Natural Watershed has an elongate shape with the highest elevation near the middle of its length. The watershed area is 29 square miles (18,600 acres) above the point of diversion. The highest elevation within the watershed is 5,176 feet above mean sea level (ft msl) and the lowest elevation is 2,765 ft msl at the point of diversion. The flow length from highest to lowest elevation is 57,398 ft. From these data, the average stream gradient is 0.04. However, the longest flow path is 76,410 ft, with an average gradient of 0.02 (Table 2-1).

Table 2-1: Amargosa Creek Watershed Characteristics

Characteristic	Value
Area (sq miles)	29
Highest point (feet above sea level, ft asl)	5,176
Lowest point (ft asl)	2,765
Flow length from highest to lowest(ft)	57,398
Highest flow path average gradient	0.042
Longest flow length (ft)	76,410
Longest flow path average gradient	0.02

Amargosa Creek is tributary to Lake Lancaster (detention basin north of Avenue H), Piute Ponds, and then Rosamond Dry Lake. The Natural Watershed area (29 square miles) is approximately 20 percent of the drainage area to Lake Lancaster (160 square miles) and approximately 2 percent of the drainage area to Rosamond Dry Lake (1,200 square miles) (Figure 2-5).



File: Palmdale\Projects\TB_WAA_7909\$Water Report\2-4 AC Watershed.mxd

NOTES:

Coordinate System: GCS North American 1983
Horizontal Datum: NAD 83

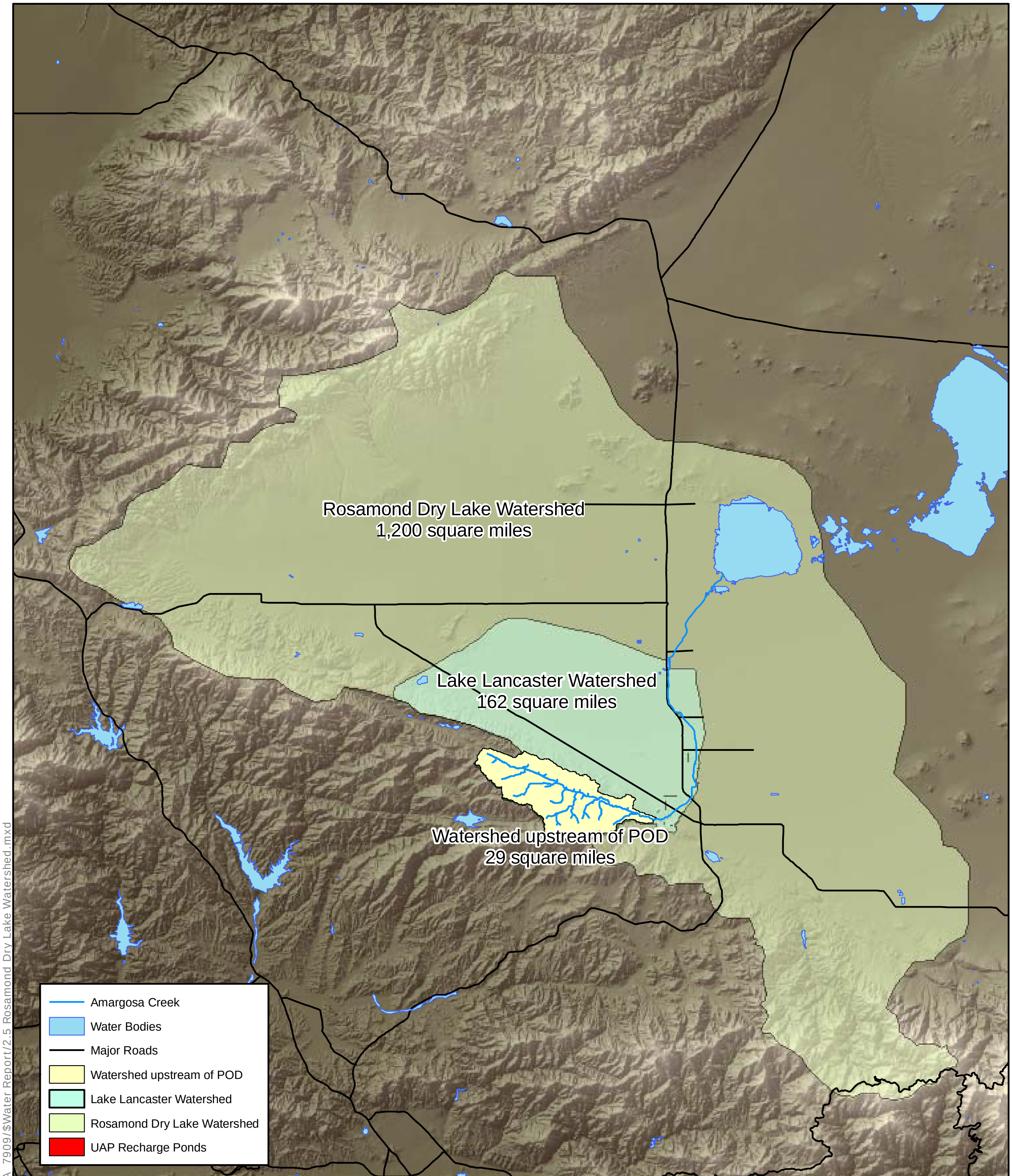
Amargosa Creek Watershed Above the Point of Diversion

0 0.5 1 2 3 4
Miles

SAIC
From Science to Solutions

DATE: 07-08-09 BY: JD

FIGURE:
2-4



File: Palmdale/Projects/WAA_7909/$Water Report/2.5 Rosamond Dry Lake Watershed.mxd

NOTES:
Coordinate System: UTM Zone 11N
Horizontal Datum: NAD 83
Rosamond Dry Lake Watershed
from French et al 2006

Rosamond Dry Lake Watershed

0 2 4 8 12 16
Miles

SAIC From Science to Solutions

FIGURE: 2.5

DATE: 06-17-09 BY: JD

2.6 CHANNEL CHARACTERIZATION

The HEC-RAS channel model is a tool to evaluate the hydraulics of a channel reach with specific characteristics of cross-section, gradient, and roughness, and is used here to evaluate the existing channel condition and the proposed channel modifications required by the Upper Amargosa Project (UAP). These cross-sections will be used in subsequent hydraulic characterizations to: determine berm elevations for the recharge facility and Nature Park; inform statements of flood risk and impacts for the UAP Environmental Impact Report; evaluate potential impacts to Public Trust Resources; and to provide hydrologic data for developing Memoranda of Understanding.

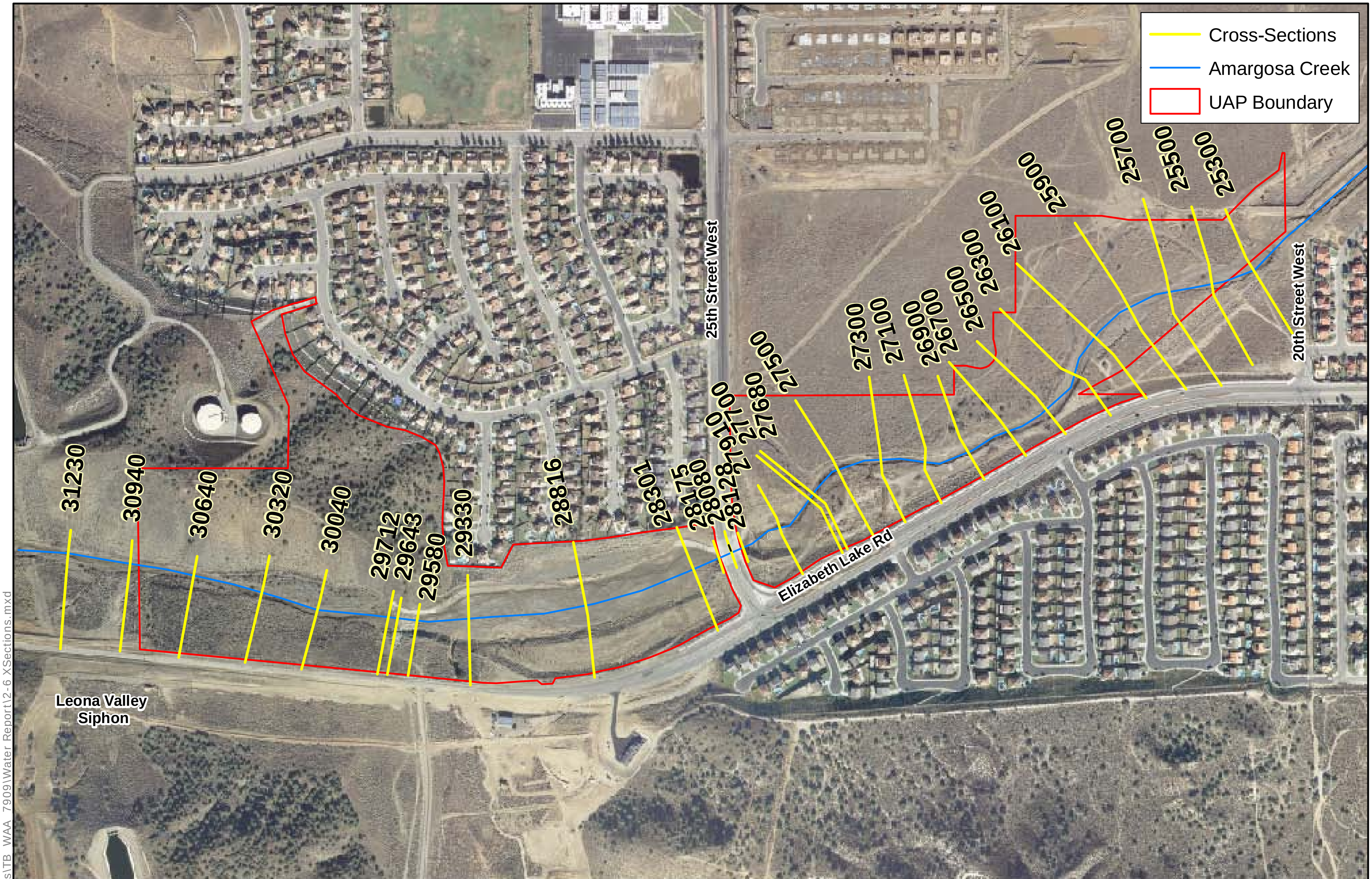
2.6.1 Amargosa Creek Channel Cross-sections

The HEC-RAS model requires channel cross-section data as an input. Cross-sections of the channel have been previously prepared (herein referred to as "pre-2003") and used in hydraulic analyses for flood control, dam operations, and bridge scour estimates (Figure 2-6). Updated cross-sections of the channel near and within the UAP boundary have been prepared using the US Army Corps of Engineers' HEC-RAS software package, January 2006 Light Detection and Ranging (LiDAR) data, and October 2008 aerially-based contour data. The previous channel cross-sections from the PACE 2004 ("pre-2003") contain a systematic error in elevations upstream of the 25th Street West Bridge. The in-channel cement rip-rap, which most likely has not changed elevation, was significantly lower in elevation in the pre-2003 cross-sections compared to the 2006 LiDAR data and 2008 contour data. An upward adjustment of 5 feet was added to the pre-2003 cross-sections upstream of the 25th Street West Bridge to correct the error.

Prior Cross-Section Data (pre-2003)

Consultant reports were reviewed for background hydraulic and morphologic information of Amargosa Creek; existing cross-section locations; and previous hydraulic calculations of flow through the Amargosa Creek channel.

SAIC obtained the HEC-RAS models used by PACE for bank stabilization between 20th Street West and 10th Street West (PACE 2003a, PACE 2003b). It is unclear how and when the original/existing cross-section data was compiled for these reports, however, it is pre-2003. PACE was retained by the City of Palmdale to complete a Letter of Map Revision (LOMR) request in order to revise FEMA's Flood Insurance Rate Map for the reach of Amargosa Creek located between the Elizabeth Lake Road crossing with Amargosa Creek and the Antelope Valley Freeway. The LOMR request was made to reflect reduced flows in Amargosa Creek due to construction of the Amargosa Dam. Cross-sections used in the hydraulic analysis are identical to the existing conditions used in prior PACE reports, but include the entire stream reach from Amargosa Dam to the freeway. SAIC was unable to obtain the HEC-RAS model used by PACE in its 2004 report, but did obtain a hard-copy of the cross-sections, and the data was manually input to HEC-RAS. It is unclear how and when the original/existing cross-section data was compiled for this report, however, it is pre-2003.



NOTES:

Coordinate System: CA State Plane Zone V (feet)
Base Map: LARIAC 4-in resolution Airphoto, 2006

UAP Channel Cross-Sections

0 250 500 1,000 1,500 2,000
Feet

SAIC From Science to Solutions

FIGURE:

2-6

DATE: 12/05/08 | BY: D Beckwith

File: T:\Palmdale\Projects\TB_WAA_7909\Water Report\2-6 XSections.mxd

From the Leona Valley siphon to the 25th Street West Bridge, the pre-2003 cross-sections from the 2004 PACE report are lower in elevation than later cross-sections, by as much as 7 feet in some areas. This includes difference in approximately five feet in elevation in-channel cemented rip-rap between cross-sections 29643 and 29580 between pre-2003 and 2006 and 2008. The pre-2003 cross-sections are also 3-4 feet lower in elevation than corresponding locations on the topographic work maps accompanying the pre-2003 cross-sections. It is possible that surveying of the cross-sections upstream of the 25th Street West Bridge was performed using an incorrect benchmark. If the pre-2003 cross sections upstream of the 25th Street West Bridge are adjusted upwards by 5 feet, the difference in elevation of the in-stream armoring between pre-2003 and January 2006, the channel shows aggradation of up to two feet above the armoring and minimal elevation change below the armoring. This adjustment is shown in Figure 2-7 as “adjusted pre-2003”. The velocities from the 2004 PACE report and the width-to-depth ratios for the cross-sections were not changed, as all cross-sections above the bridge were shifted the upwards the same amount and the slope between cross-sections was not changed.

Updated Cross Sections (2006 and 2008)

LiDAR data collected by the Los Angeles Region Imagery Acquisition Consortium (LAR-IAC) during the period December 2005 to March 2006 was provided to SAIC by the City of Palmdale. The data is suitable for creating 2-ft contours and has a vertical accuracy in open and scrub terrain of 0.7 ft, and 0.87 ft, respectively (Dewberry 2006). The LiDAR data for Amargosa Creek was recorded in January 2006 and used to create a triangulated irregular network (TIN) of the topology of the UAP representative of ground surface conditions for January 2006.

In October 2008, the Azimuth Group provided topography data for the existing conditions at the UAP site. The data was provided in contour format with a vertical accuracy of approximately three (3) inches. The contour data was sorted to include only the measurements of ground surface (e.g. elevation measurements representing trees and walls were removed), combined with spot elevation data, and used to create a TIN of the UAP representative of ground surface conditions for October 2008.

Cross-sections of the Amargosa Creek channel were positioned at locations representative of similar channel morphology throughout the channel, at channel structures, and at locations where changes occur in channel slope, shape, and roughness. The cross-sections were positioned as close as possible to cross-section locations used in the 2004 PACE report in order to provide a comparison of morphologic and conveyance changes between pre-2003 and January 2006/October 2008. On average, each cross-section is within 19.8 ft from its corresponding PACE cross-section (std dev 12.2 ft, n=29). HEC-GeoRAS was used to extract channel geometry from each TIN at the cross-section locations and to prepare the cross-sections for use in HEC-RAS.

The channel geometry was adjusted at several cross-sections to better represent current and near-future channel conditions. The channel geometry under the 25th Street West Bridge as defined in PACE 2004, was used in place of TIN-extracted data because LiDAR and contour data only registers the top-most land surface. The channel geometry of cross-sections at the most downstream end of the UAP near the proposed 20th Street West Bridge was replaced with WEST 2006 data to better represent likely future channel conditions post bridge construction.

Bank stations at each cross-section were defined for the January 2006 and October 2008 cross-sections using data in the corresponding PACE 2004/WEST 2006 cross-sections. Manning’s *n* values for the channel and overbanks were taken directly from the corresponding PACE/WEST HEC-RAS models.

2.6.2 Amargosa Creek Morphologic Changes

In general, the Amargosa Creek channel from the Leona Valley siphon to the 25th Street West Bridge experienced minor morphologic changes between pre-2003 and October 2008. The minimum channel elevations raised up to two (2) feet between pre-2003 and January 2006 from the Leona Valley siphon (cross-section 31230) to the 25th Street West Bridge (cross-section 28128), lowered by one to two feet below the bridge for one thousand linear feet of channel, and changed little from 1000 feet downstream of the 25th Street West Bridge (cross-section 27100) to the 20th Street West (cross-section 25300) (Figure 2-6). No significant changes in minimum channel elevation occurred between January 2006 and October 2008 (Figure 2.7).

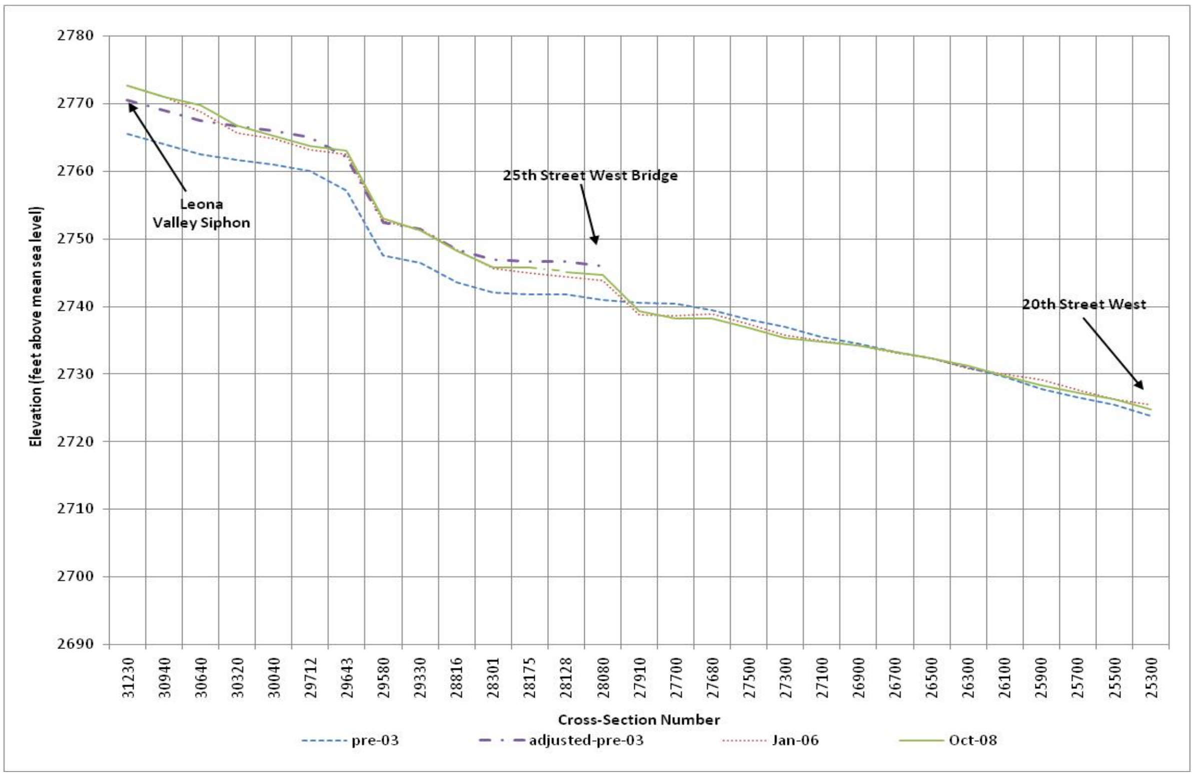


Figure 2-7: Minimum Channel Elevations for pre-2003, January 2006, and October 2008.

The width to depth ratio did not change significantly from the Leona Siphon to the 25th Street West Bridge (Figure 2-8 and 2-9, paired two-sided t-test, alpha=0.05, t=0.089, df=11, p=0.931). From the 25th Street West Bridge to 20th Street West, however, between pre-2003 and January 2006 the channel became significantly more narrow and deeper (Figure 2-8 and 2-9 paired two-sided t-test, alpha=0.05, t=4.351, df=15, p=0.001). Between January 2006 and October 2008 the width to depth ratio from the 25th Street West Bridge to 20th Street West did change slightly in a few locations, but not significantly overall (Figure 2-8 and 2-9, paired two-sided t-test, alpha=0.05, t=-2.022, df=15, p=0.06).

The narrowing and deepening of the channel most likely occurred in December 2004 - January 2005 when Amargosa Creek watershed experienced high rainfall and a large flow passed through Amargosa Creek channel. Downstream of the 25th Street West Bridge the channel also has been straightened between cross-sections 27100 and 26300 with the re-alignment of Elizabeth Lake Road.

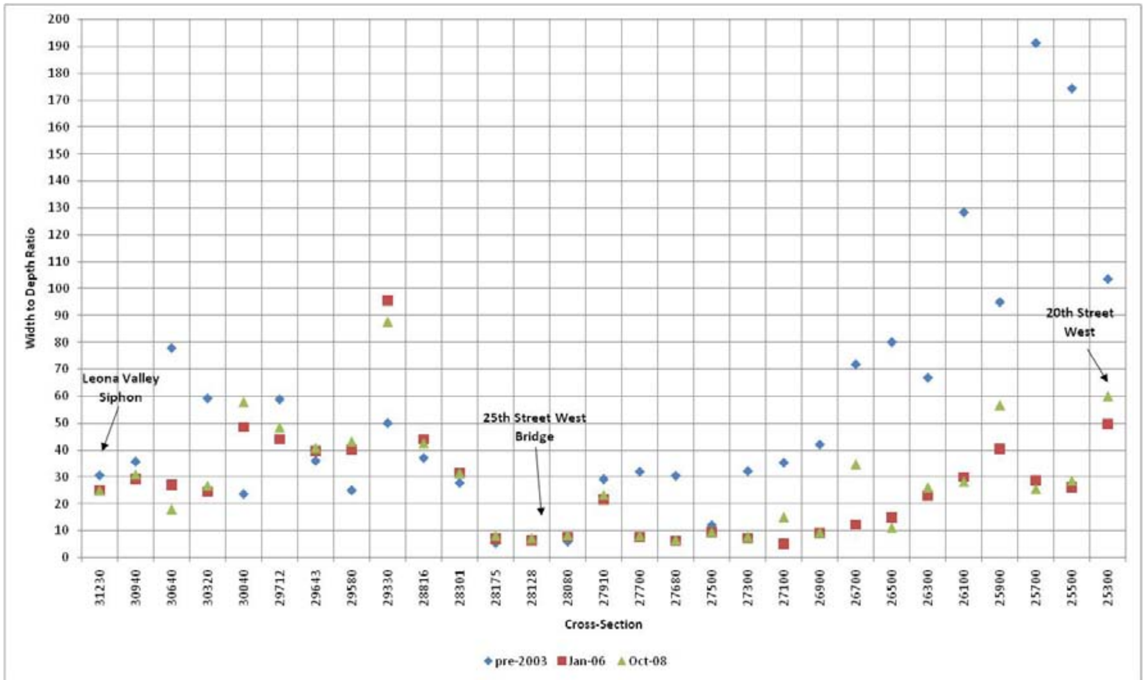


Figure 2-8. Width to Depth Ratio for pre-2003, January 2006, and October 2008.

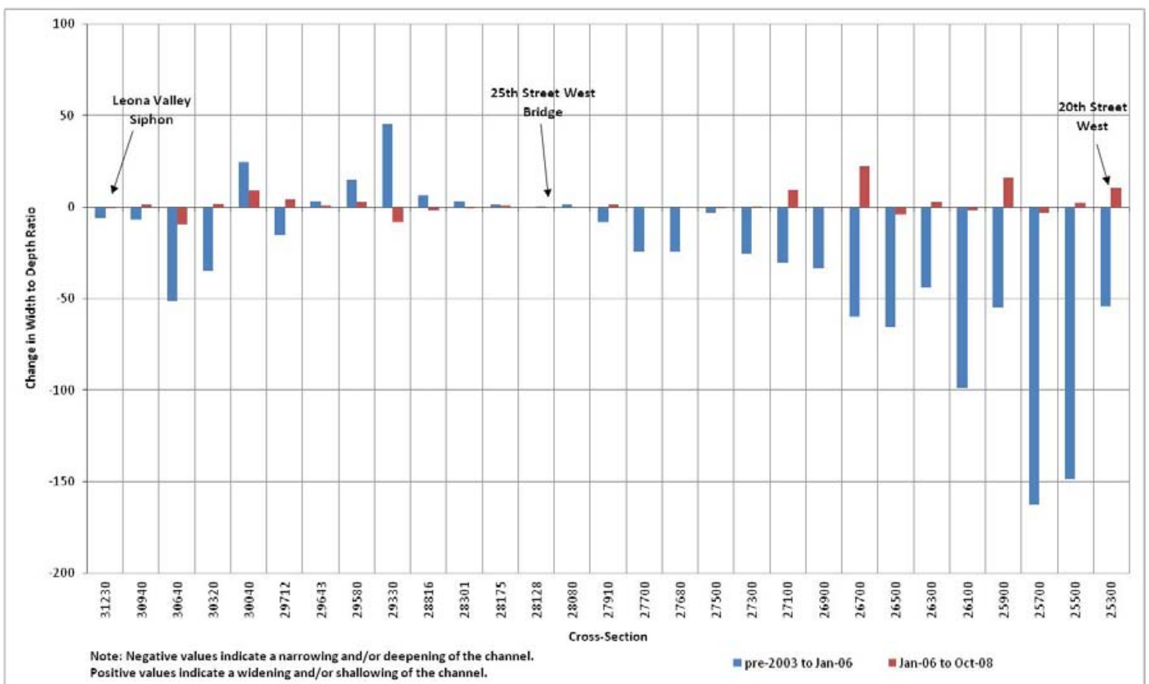


Figure 2-9: Change in Width to Depth Ratio between pre-2003 and January 2006 and between January 2006 and October 2008.

2.6.3 HEC-RAS Model

The US Army Corps of Engineers' HEC-RAS software package is designed to perform one-dimensional hydraulic calculations for a full network of natural and constructed channels. The basic computational procedure is based on the solution of the one-dimensional energy equation. Energy losses are evaluated by friction (Manning's equation) and contraction/expansion (coefficient multiplied by the change in velocity head). The basic equations used are the discharge formula and manning's equation.

Discharge formula

$$Q = AV$$

where:

Q = Discharge ([ft^3/s];
A = Cross-sectional area ([ft^2];
V = Average linear velocity (ft/s), Manning's equation;

Manning's equation

$$V = \frac{k}{n} R_h^{\frac{2}{3}} \cdot S^{\frac{1}{2}}$$

where:

V = Cross-sectional average velocity (ft/s);
k = Conversion constant equal to 1.486 ($ft^{1/3}/s$) for U.S. customary units;
n = Manning coefficient of roughness (independent of units);
R_h = Hydraulic radius (ft);
S = Slope of the water surface or the linear hydraulic head loss (ft/ft).

A HEC-RAS steady-flow simulation of the 100-year flood was performed with the pre-2003, January 2006, and October 2008 cross-sections. From the 25th Street Bridge West to 20th Street West where the channel has become narrower, the cross-sectional area of flow for the 100-year flood has decreased and flow velocity has increased compared to pre-2003 values (Figure 2-10, Figure 2-11).

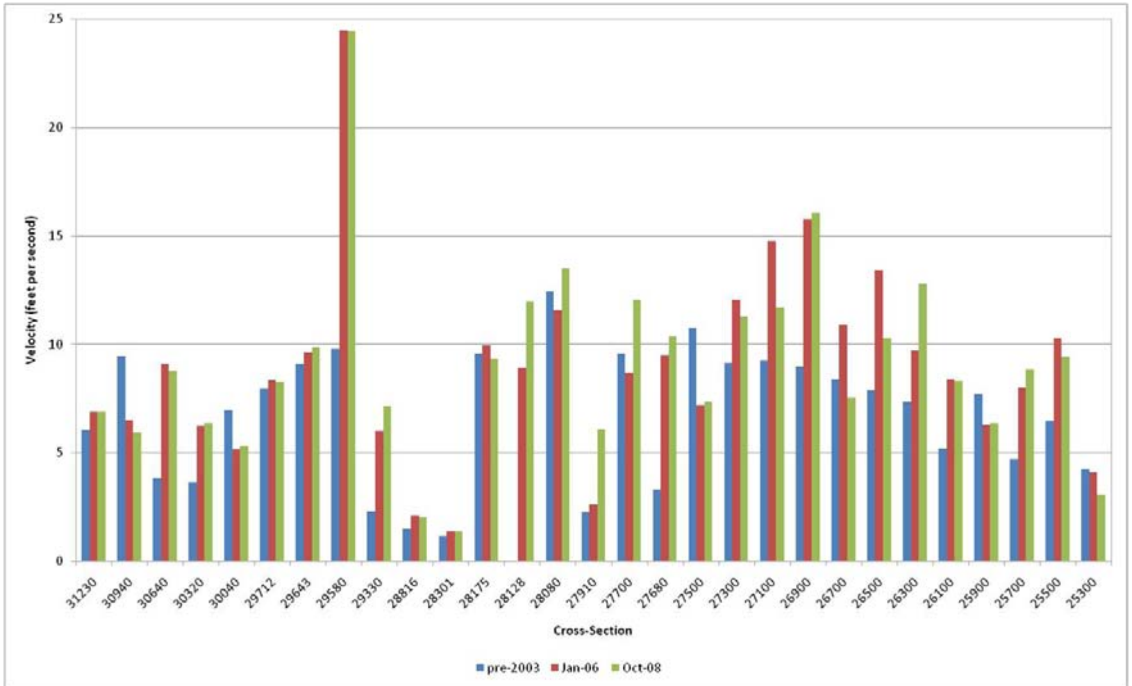


Figure 2-10: Channel Velocity for pre-2003, January 2006, and October 2008 Under Equivalent Hydrographs.

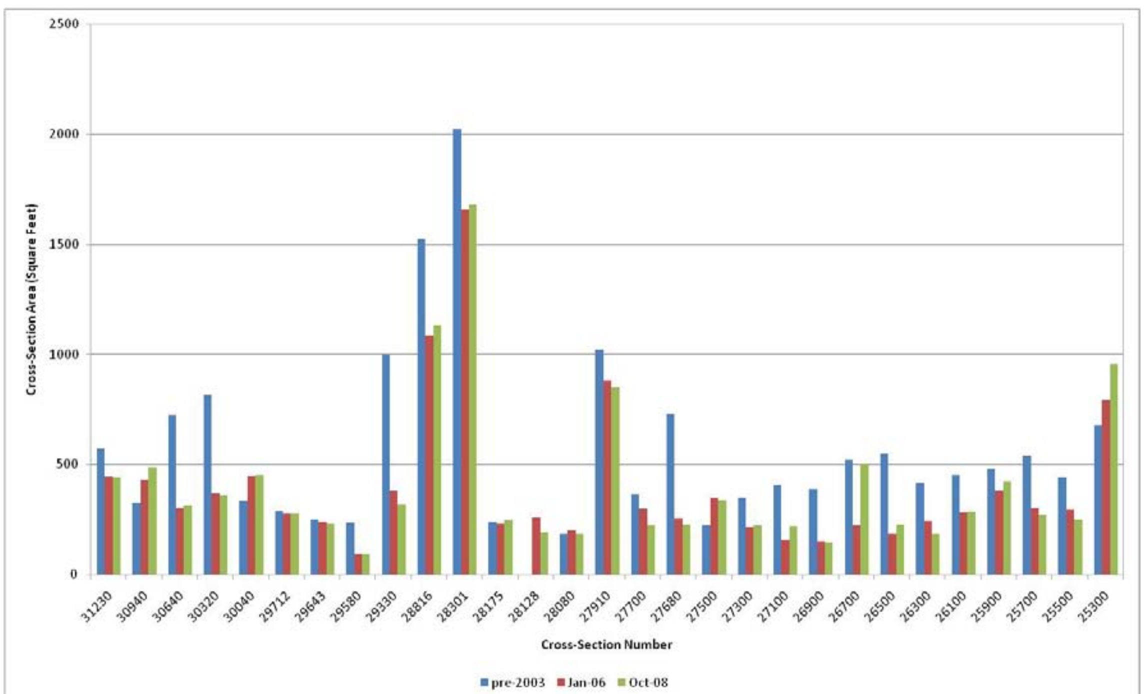


Figure 2-11: Flow Area for pre-2003, January 2006, and October 2008 Under Equivalent Hydrographs.

2.7 AQUIFER SYSTEM

A cross-section of the subsurface geology underneath Amargosa Creek from the UAP to Rosamond Dry Lake was prepared from available well logs and borings and shows the dominant geologic features including the significant aquifers and aquitards (Figure 2-12). The principal aquifer is unconfined alluvium mostly composed of unconsolidated sand and gravel that overlies ancient lake bed deposits. The closest well log near the UAP (approximately ½ mile downstream) shows that bedrock occurs 285 ft bgs (2425 feet elevation). Further downstream (1¼ mile downstream of the UAP) bedrock occurs at 700 ft bgs (1910 feet elevation), suggesting that the bedrock dips steeply from the southwest to the northeast. The unconfined principal aquifer reaches depths of 800 ft bgs downstream from the UAP. Below the unconfined alluvium occurs a series of clay layers deposited as an ancient lake bed with thickness ranging from 100 to 300 ft. The deep aquifer is confined below the ancient lake bed deposits and its depth is unknown. Approximately ten miles downstream of the UAP evidence of middle and upper lake beds occur in the lithologic logs. Near Avenue J, silts and clays begin to dominate the surface sediments based on the borehole data and the boundary of the playa deposits mapped by Ponti and others (1981). The silts and clays are less permeable and impede seepage in the Amargosa Creek channel bed downstream of Avenue J.

An understanding of the geologic setting is fundamental to the understanding of the disposition of streamflow that occurs in the Amargosa Creek, as well as any water imported to and stored within the principal aquifer of the Antelope Valley groundwater basin.

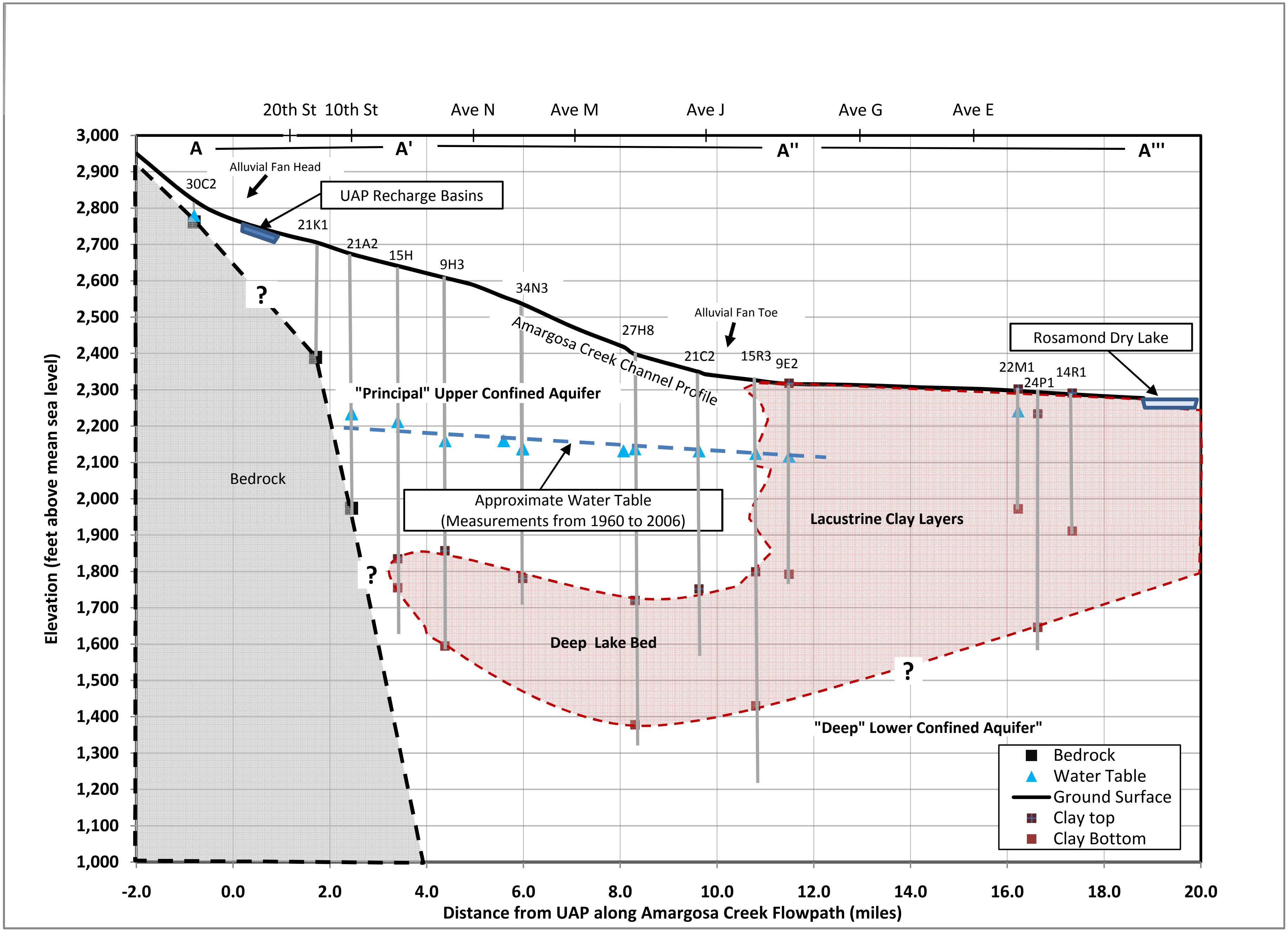


Figure 2-12: Subsurface Geologic-Hydrologic Cross-section Upper Amargosa Project

3 HYDROLOGY

Rainfall percolates through the vadose zone to recharge the groundwater aquifer; it entrains and transports constituents that ultimately impact the quality of water in storage within the groundwater basin. The following sections present the analyses of rainfall, streamflow data, groundwater elevations, and groundwater quality.

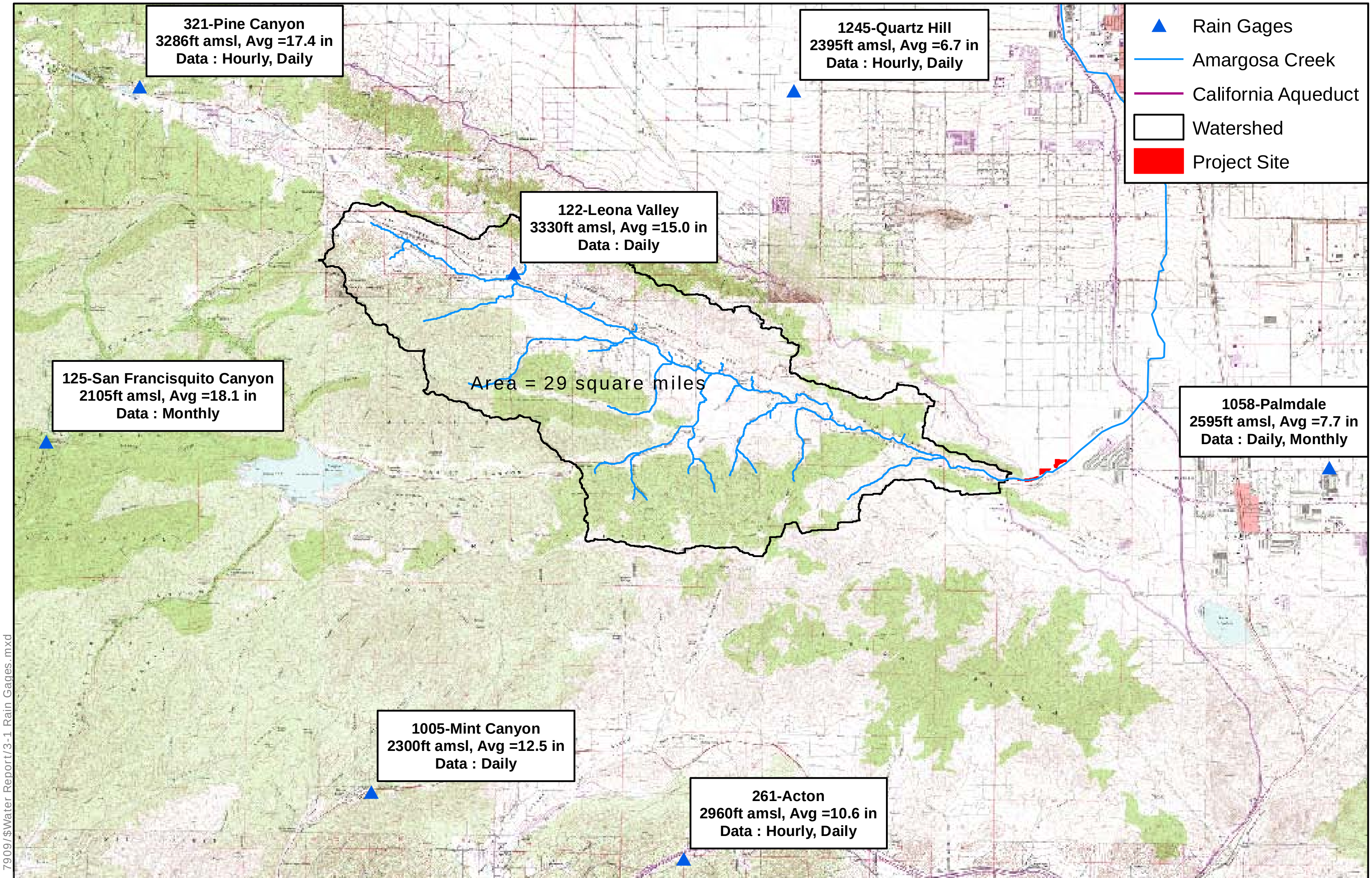
3.1 RAINFALL

An understanding of the rainfall patterns in space and time is fundamental to the understanding of the streamflow that occurs in the Amargosa Creek. Most of the rainfall in the Amargosa Creek watershed occurs from mid-latitude Pacific cyclonic storms during the winter. The following sections present the available rainfall data and characterize the rainfall patterns in the area of the UAP.

3.1.1 Data Source

Rainfall is typically recorded in all the rain gages in the region during storm events. Seven rainfall records are available from the Los Angeles County Department of Public Works (LACDPW 2008). The data are available in hourly, daily, and monthly formats. There are six gages surrounding Amargosa Creek watershed and one gage inside the Amargosa Creek watershed (Figure 3-1 and Table 3-1). One gage (122 - Leona Valley) is located within the Natural Watershed and the remaining six gages are located within eight miles surrounding the Natural Watershed. The data collected at the Leona Valley gage (#122) best represents the rainfall occurring in the Natural Watershed and will be the focus of the following rainfall analyses.

Rainfall has been recorded at several intervals, hourly, daily, and monthly; and the period of record varies from 10 years to 90 years (Table 3-1). Gage elevations vary within 1225 feet, ranging from 2105 feet above mean sea level (ft msl) to 3330 ft msl. The average annual rainfall for the period of record from each gage demonstrates a strong west to east decreasing gradient ranging from 18 inches each year in/yr to 7 in/yr typical of a rainshadow effect on the leeward of a mountain range (Figure 3-1). Rainfall records for location 122 – Leona Valley end in 1992, while all other records are available to within a few years of the present.



NOTES:
Base Map: USGS Quadrangles
Ritter Ridge, Del Sur, Sleepy Valley, and Lancaster West
Coordinate System: UTM Zone 11N
Horizontal Datum: NAD 83

N

Amargosa Creek Watershed
Rain Gages

0 0.5 1 2 3 4
Miles

From Science to Solutions

DATE: 10/03/08
BY: J. Degner

FIGURE:
3-1

File: WTR-Palmdale/WAA_7909/$Water Report/3-1 Rain Gages.mxd

Table 3-1: Rain gages

Station Number	Station Name	Measurement Interval [Period of Record]	Latitude	Longitude	Gage Elevation (ft msl)	Average Period of Record (in/yr)
122	Leona Valley	Daily [1929 to 1992]	34.6311	-118.3228	3330	15.0
125	San Francisquito Canyon	Monthly [1919 to 2007]	34.5903	-118.4542	2105	18.1
261	Acton - Escondido	Daily [1897 to 2008] Hourly [1996 to 2008]	34.4950	-118.2728	2960	10.6
321	Pine Canyon Control Station	Daily [1931 to 2008] Hourly [1997 to 2008]	34.6733	-118.4292	3286	17.4
1005	Mint Canyon Fire Station	Daily [1946 to 2005]	34.5097	-118.3611	2300	12.5
1058	Palmdale	Monthly [1931 to 2007] Daily [1953 to 2008]	34.5881	-118.0919	2595	7.7
1245	Quartz Hill	Daily [1986 to 2007] Hourly [1998 to 2008]	34.6744	-118.2444	2395	6.7
NCEP/EMS Multi-Sensor Radar Estimated Rainfall		Hourly [1996 to present]	NA	NA	NA	NA

3.1.2 Rainfall Characterization

Rainfall in the region was characterized by evaluating data collected at various locations. The following sections present rainfall characterizations based on several techniques.

3.1.2.1 Double Mass Analysis

Double mass analysis is a technique commonly employed to determine corrections to hydrometeorological data to account for changes in data collection procedures or other local conditions. The changes may result from a variety of things including changes in instrumentation, changes in observation procedures, or changes in gage location or surrounding conditions. Double mass plots present the daily rainfall totals for two stations with overlapping records (Figure 3-2). Break points are locations of slope change in the double mass plot that indicate an inconsistency between two gages. Correction factors can be computed for the break points to alter the slope of segments of the double mass plot. The double mass plots comparing station 122 and station 1058, and stations 321 and station 1058 contained a slight curvature in slope, and may indicate that conditions at station 1058 have changed over time thus the record may contain inconsistent data, which may warrant further investigation if this data is used in the analysis. Station 1245 double mass curves contain flat slopes which indicate missing data. All other double mass plots show no significant changes in slope. All data records are generally consistent with a slight indication of potential inconsistency in the rainfall record from Station 1058 - Palmdale.

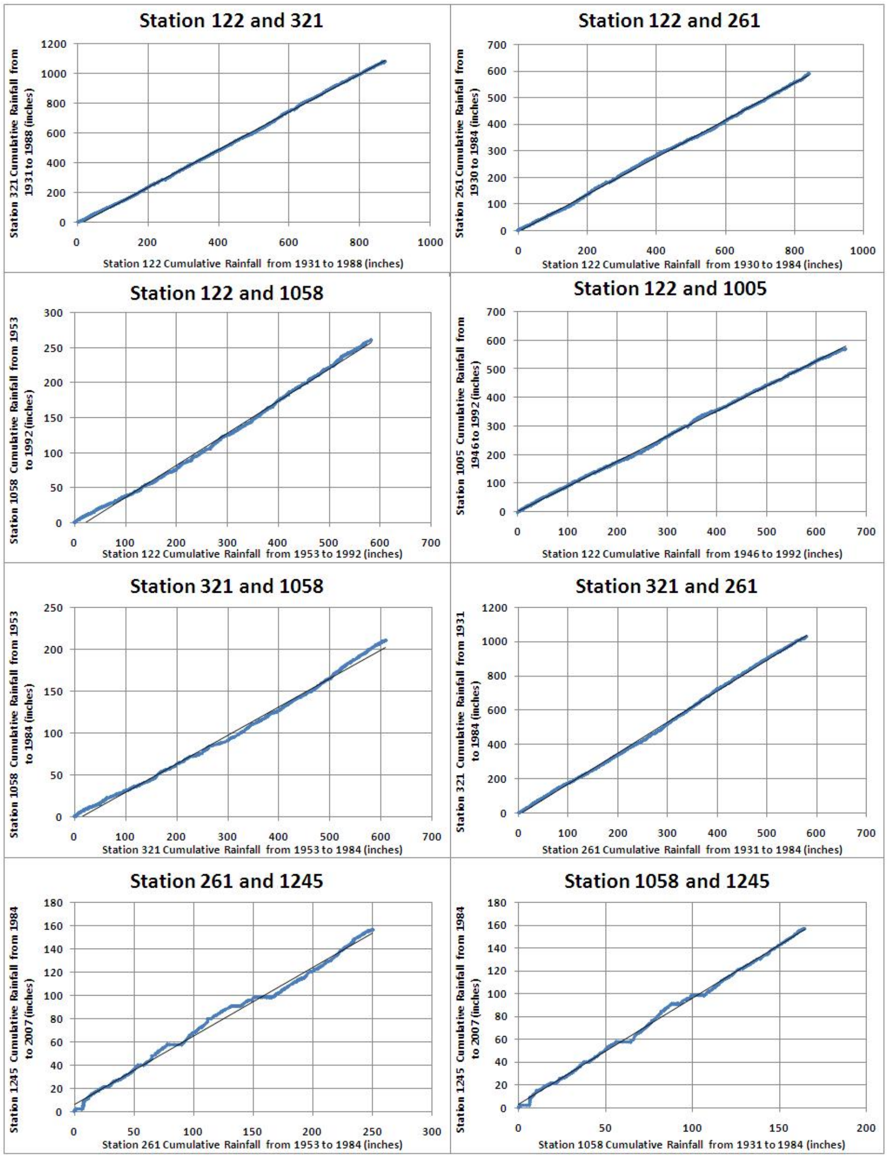


Figure 3-2: Double Mass Curves using Daily Rainfall Totals

3.1.2.2 Cumulative Probability

The cumulative probability distribution was computed to understand the likelihood of daily rainfall events of a certain magnitude. The cumulative probability is the probability of rainfall less than or equal to that amount. Cumulative probabilities were developed for the all the stations with daily rainfall records and graphed (Figure 3-3). On average, rainfall occurs on 38 days each year in the mountains to 26 days each year in the valley. The daily rainfall on average exceeds 1 inch on six days each year in the mountains and 2 days each year in the valley.

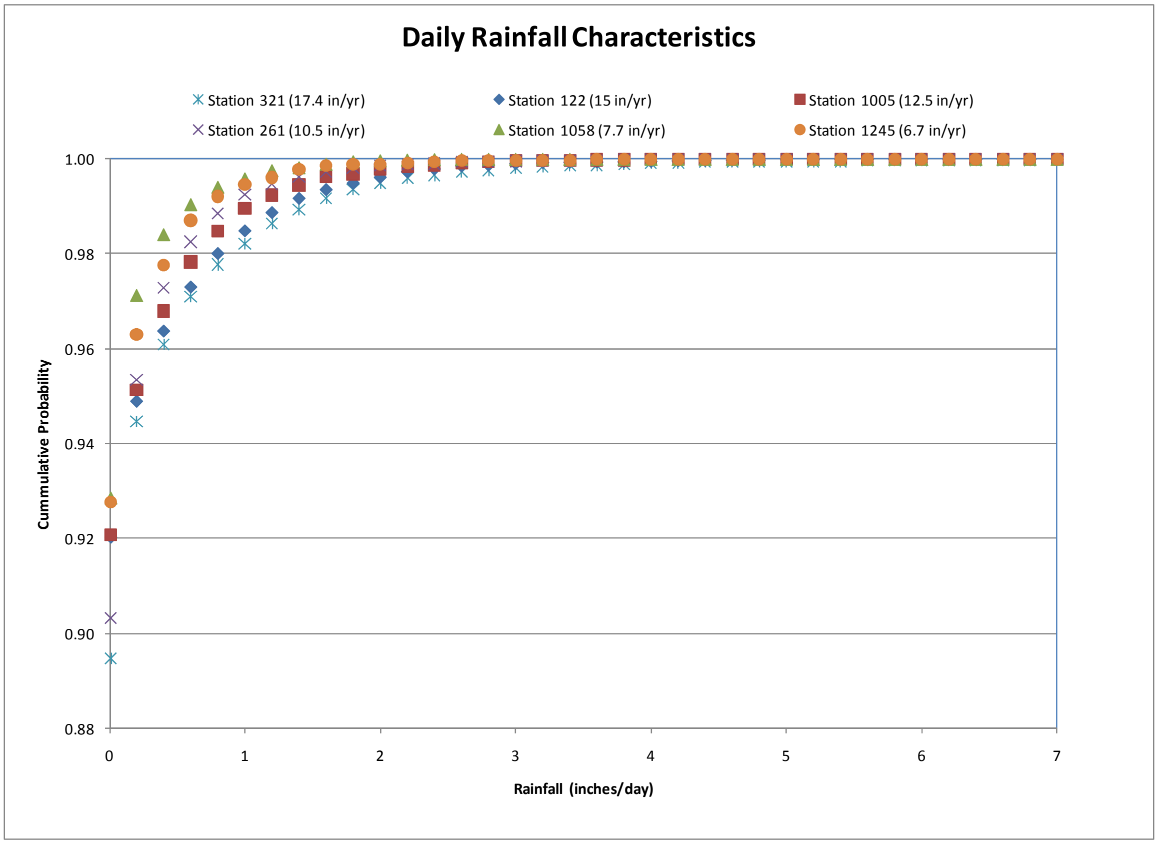


Figure 3-3: Cumulative Probability of a daily rainfall amount from each of the six rainfall records.

3.1.3 Estimating Hourly Rainfall

Rainfall rates during a storm event control the runoff from the Natural Watershed that occurs in the Amargosa Creek at the UAP. Hourly rainfall data would provide a more accurate estimate of runoff from the Natural Watershed than rainfall data reported at daily intervals. Hourly rainfall totals have been measured at Stations 321 and 261 for most of the base period (WY 1997- 2007), with some gaps in the records. Station 122 is the only gage located within the Natural Watershed, but no hourly rainfall record is available. The hourly rainfall was estimated at Station 122 from the two hourly rain gage records (261 and 321) by scaling the available hourly data with the ratio of the long-term mean of Station 122 and Station 321 or 261. Time lags in the rainfall recorded were noted at the hourly interval between station 321 and 261 that are not noticeable in the daily totals. For this reason, the estimated hourly rainfall may not accurately represent the actual rainfall that occurred at a specific hour, but the estimate does accurately represent the magnitude and temporal structure of storms events for the base period critical to hydrologic analyses. The estimated hourly rainfall at Station 122 is a synthesized rainfall record.

The mean annual rainfall for the Station 122 synthesized record is 15.6 inches as computed from WY 1997 - 2007. The mean annual rainfall during the period of record for Station 122 is 15.0 inches. The slightly higher value for the estimated hourly rainfall is consistent with period from WY 1997-2007 being slightly wetter than the period of record for Station 261. The annual rainfall totals for the estimated data is consistent with the rainfall totals measured at Station 261 and 321 (Figure 3-4).

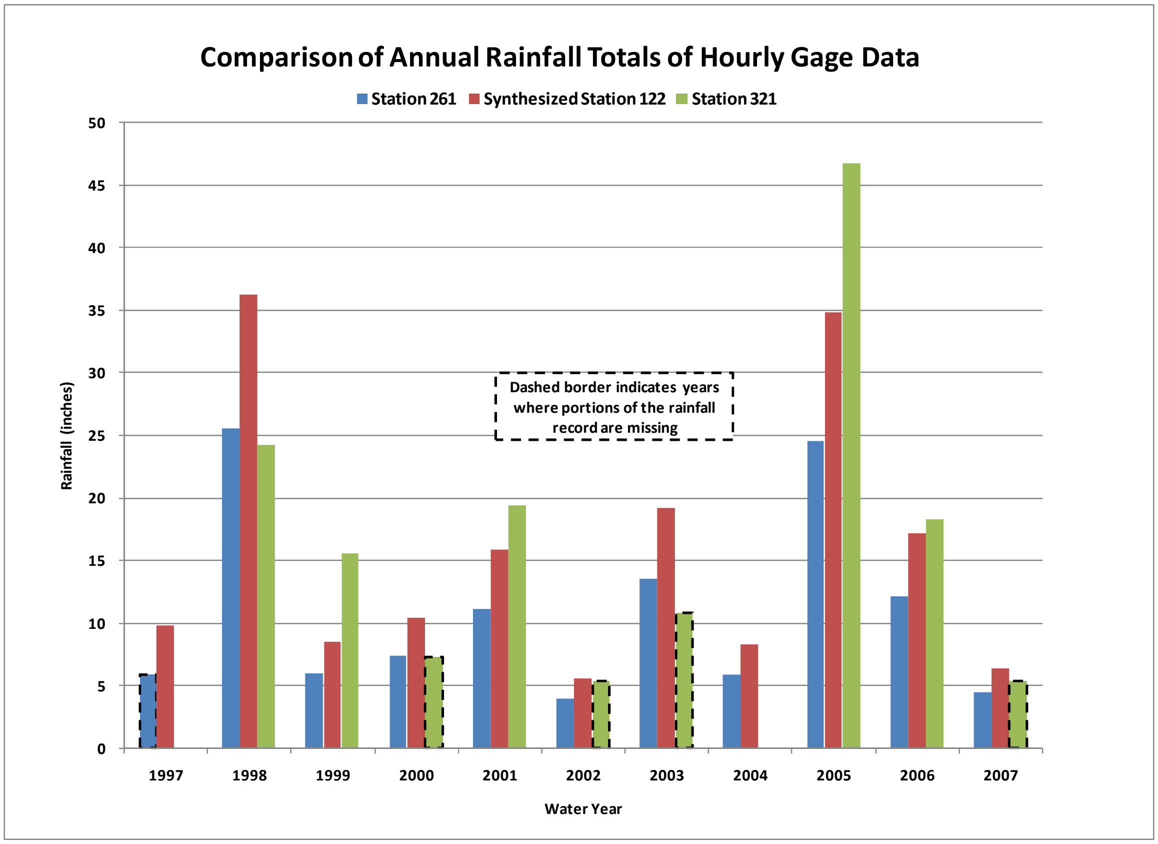


Figure 3-4: Comparison of Annual Rainfall Totals

The cumulative probability distribution of hourly rainfall totals was computed from the synthesized rainfall record at Station 122 and compared to the measured data from Station 321 and Station 261 (Figure 3-5). The comparison of hourly rainfall represents the strong west to east decreasing gradient observed in the daily data (Section 3.1.1). Houly rainfall is expected to exceed 0.2 inches, 23 hours each year; and 0.5 inches, 2 hours each year, based on the cummulative probability distribution.

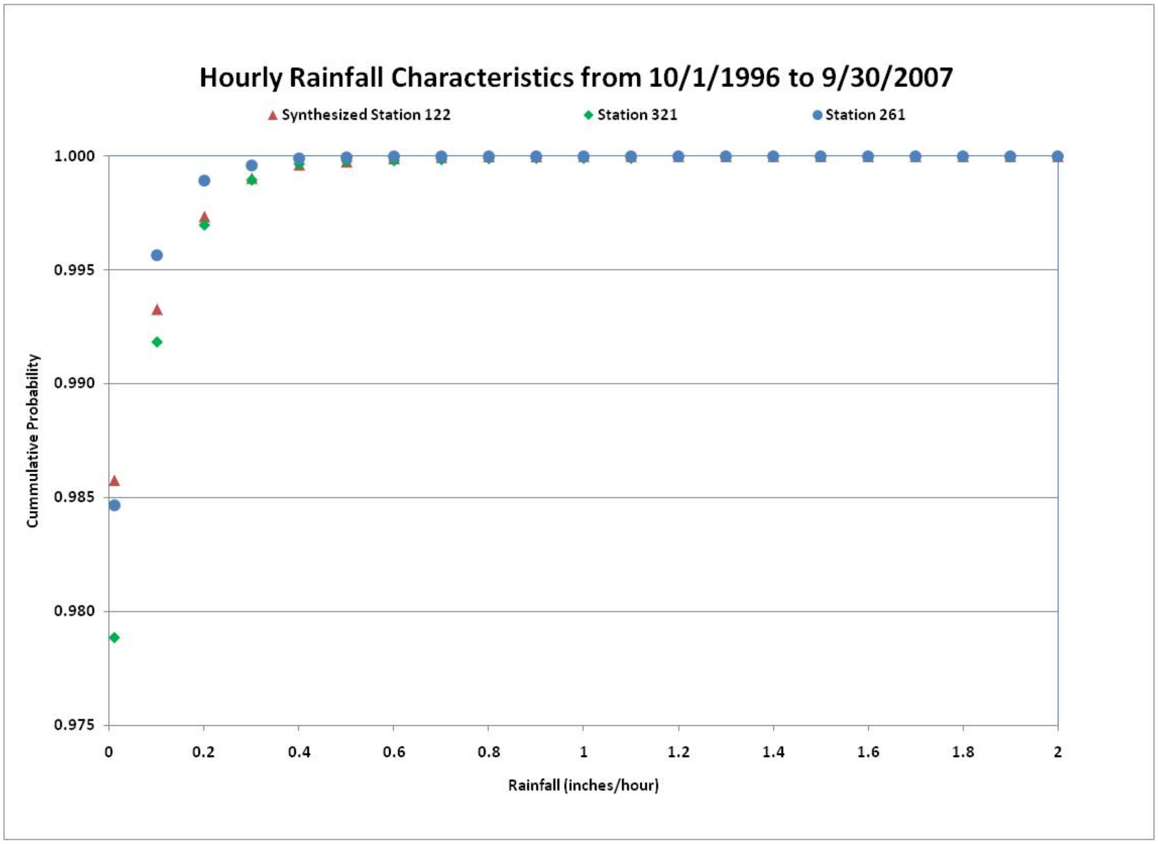


Figure 3-5: Hourly Rainfall Characteristics

3.2 SURFACE WATER HYDROGRAPHS

USGS previously installed two stream gage stations to collect data from tributaries of Amargosa Creek, 10264520 Amargosa Creek Tributary near Leona Valley and 10264530 Pine Creek near Palmdale. Amargosa Creek Tributary is a partial record station with a drainage area of 0.05 square miles where annual peak flows were measured intermittently from 1959 to 1987 (Figure 3-6).

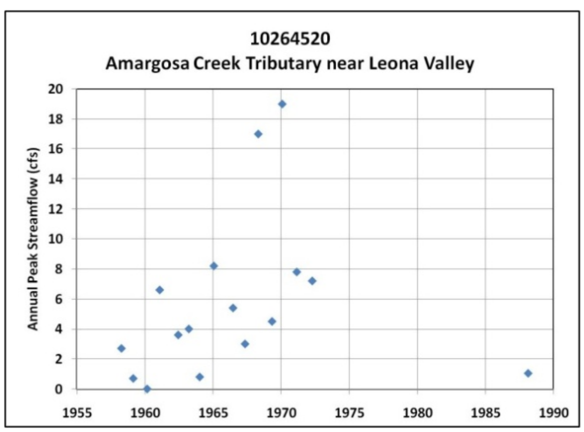


Figure 3-6: USGS 10264520 Annual Peak Streamflow

Pine Creek is a partial record station with a drainage area of 1.78 square miles with annual peak flow measurements by a crest-stage gage from 1958 to 2006 (Figure 3-7) and continuous daily mean streamflow measurements from 1988 to 1994 (Figure 3-8).

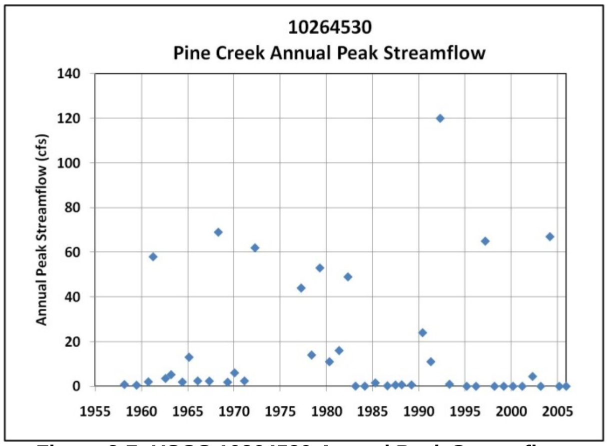


Figure 3-7: USGS 10264530 Annual Peak Streamflow

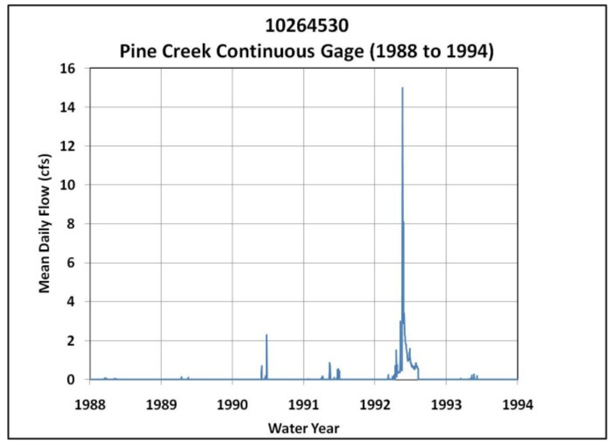


Figure 3-8: USGS 10264530 Mean Daily Streamflow

There are no surface water quality measurements from Amargosa Creek, but the hydrology (ephemeral, intermittent flows occurring with intense rainfall) would indicate runoff with low dissolved solids concentrations. The water quality of the SWP ranges from 124 to 368 mg/L with an average of 233 mg/L, below the range of 500 to 900 mg/L of TDS for water quality observations made since 2000.

3.3 GROUNDWATER ELEVATIONS

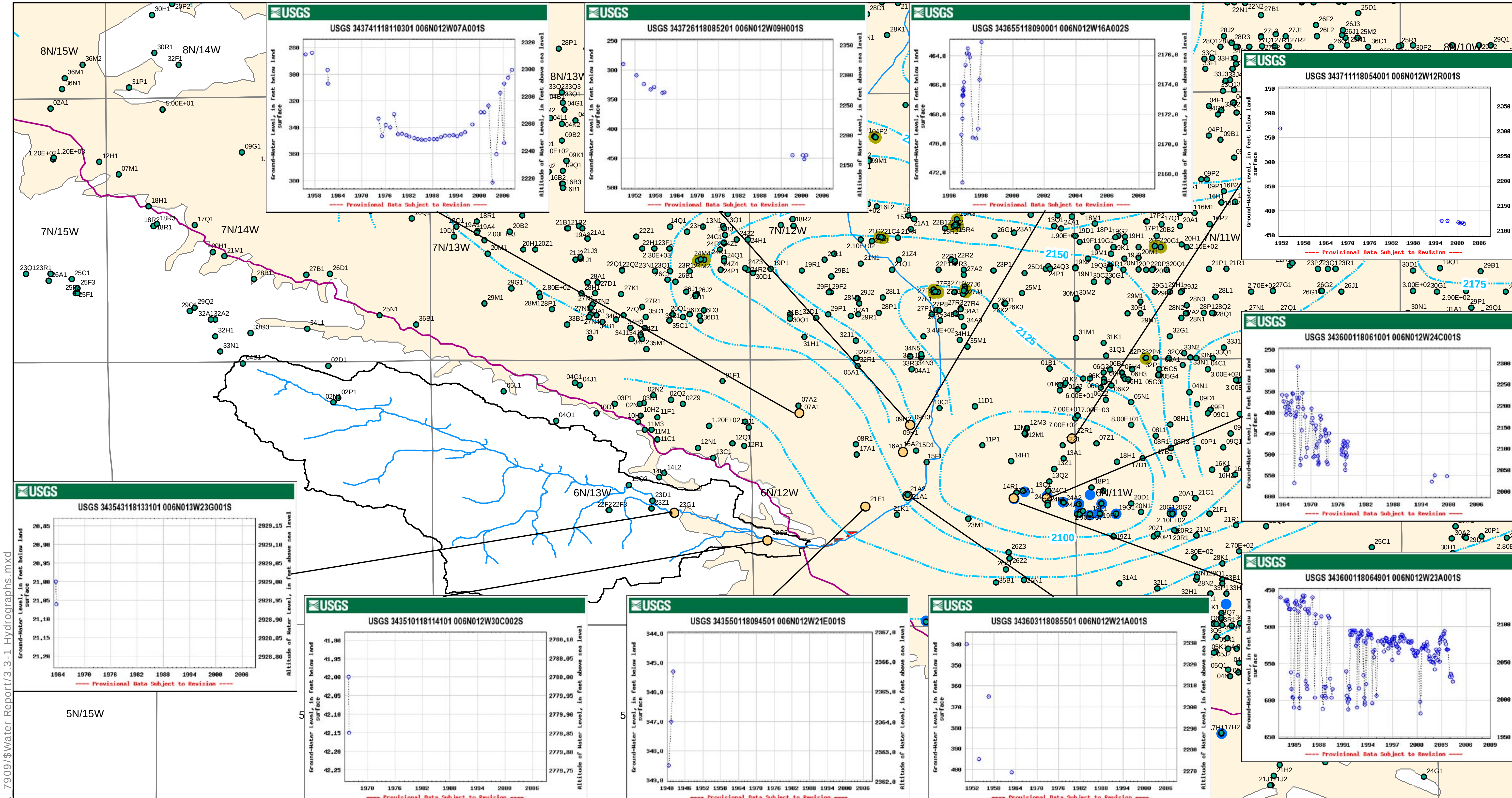
The available groundwater elevations measurements were obtained from USGS. The water level data were reviewed and the ten representative hydrographs of the water levels near the UAP are displayed on Figure 3-9. The water table presented in the cross-section (Figure 2-12) reflects recent data and is the lowest water level on record for each well from 1960 to 2006

3.3.1 Natural Watershed

The numerous seeps and sag ponds in the San Andreas Fault Zone valley floor are evidence of a shallow water table. Historical depth-to-groundwater measurements range from 4 to 42 ft bgs in the Natural Watershed. The California Geologic Survey designates this area as a seismic hazard zone for liquefaction (CGS 2005). Shallow groundwater observations suggest that little available storage capacity exists within the thin veneer of sediments in the Natural Watershed. Rainfall resulting as runoff may often be rejected from local storage in the alluvium thus producing streamflow down-channel and to the Antelope Valley.

3.3.2 Urban Watershed

Amargosa Creek crosses onto the Antelope Valley floor at the UAP. The most recent depth-to-groundwater measurements range from 420 to 450 ft bgs in the Urban Watershed approximately 2 miles down-channel from the UAP. Water that infiltrates in the stream bed and the proposed recharge basins would percolate to the water table and recharge the Lancaster subunit of the Antelope Valley Groundwater Basin. The Lancaster subunit is the largest source of groundwater in the region and most economically important subunit. Historically it has been considered comprised of two units, the "principal" and "deep" aquifer.



File: File: Palmdale/TB WAA 7909/$Water Report/3.3-1 Hydrographs.mxd

NOTES:
Coordinate System: GCS North American 1983
Horizontal Datum: NAD 83
Data collected from USGS groundwater database

2005 Groundwater Elevation Contours and Well Hydrographs

0 0.5 1 2 3 4
Miles

USGS Wells

- Wells
- Well with Graphs
- Waterworks District No. 40 Wells
- PWD Wells
- Amargosa Creek
- California Aqueduct
- 2005 Groundwater Elevations (ft asl)
- Proposed Recharge Basins
- Amargosa Creek Watershed
- Township and Range
- Adjudicated AV Groundwater Basin

SAIC
From Science to Solutions

FIGURE:
3-9

DATE: 07-15-08
BY: ???

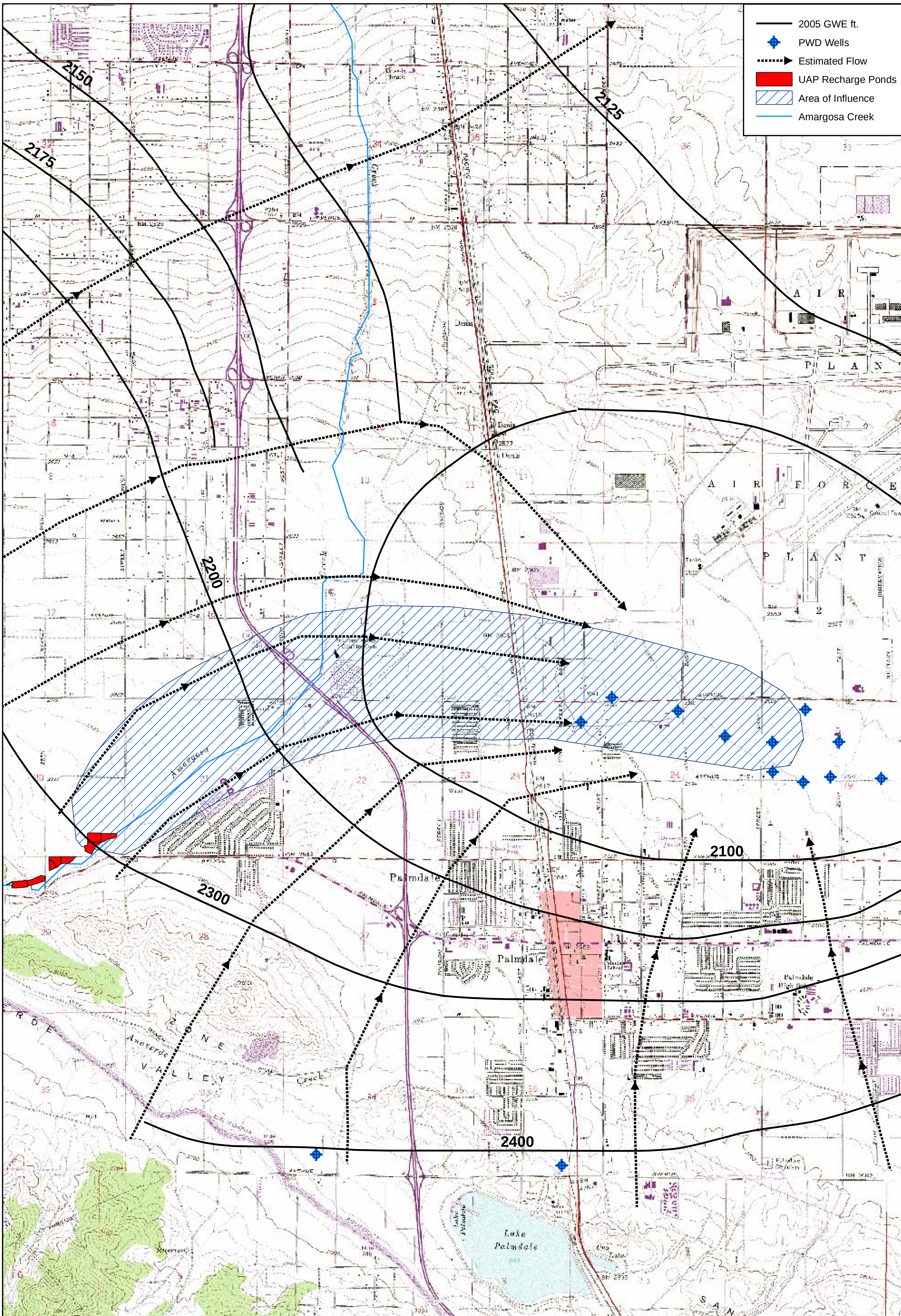
3.3.2.1 Groundwater Contours

Groundwater contours were developed using groundwater elevation measurements from the year 2005 (Figure 3-9). A groundwater depression to the north and east of the UAP exists. Based on these contours, the groundwater flow direction from the UAP is to the north and east towards the City of Lancaster and Plant 42 (Figure 3-10).

3.4 GROUNDWATER QUALITY

Water quality data were compiled from the Department of Health Services and the USGS. Overall, there are six wells in the vicinity of the proposed project with water quality data (Figure 3-11). Water quality data exceed the Secondary Maximum Contaminant Level (MCL) for Total Dissolved Solids (500 mg/L) in 15 of the 30 measurements, exceed the MCL for Nitrates (45 mg/L) in no measurements, and exceed the MCL (10 mcg/L) for Arsenic in no measurements.

Total Dissolved Solids (TDS) concentrations range from 129 to 980 milligrams per liter (mg/L) (since 2000 from 500 to 980 mg/L), nitrate concentrations (as NO_3) range from 0 to 17 mg/L and arsenic concentrations range from 0 to 3 micrograms per liter (mcg/L). Data was collected from three locations since 2000 (6N/12W-9H3, 1900301-001, and 1900803-001). The trend at the three locations is that TDS concentrations have increased coincidently with declining groundwater elevations. Generally, elevated arsenic concentrations have been encountered in the deep aquifer underneath the dry lake beds north of the UAP (AV IRWMP 2007). Nitrates (as NO_3) concentrations are thought to be residual from the historic agriculture in the region, but concentrations are well below the Maximum Contaminant Level (MCL) standard of 45 mg/L for nitrate (as NO_3). No data suggests groundwater under the UAP has degraded water quality of arsenic and nitrates experienced elsewhere in the Antelope Valley groundwater basin, and TDS concentrations could be improved with increased recharge from local surface water and SWP at UAP**.**



NOTES:

Coordinate System: UTM Zone 11N
Horizontal Datum: NAD 83

Topo map - USGS 24K Quads

Groundwater Contours from 2008 Problem Statement for the Antelope Valley Area of Adjudication

N

Upper Amargosa Project
Area of Influence

0 0.25 0.5 1 1.5

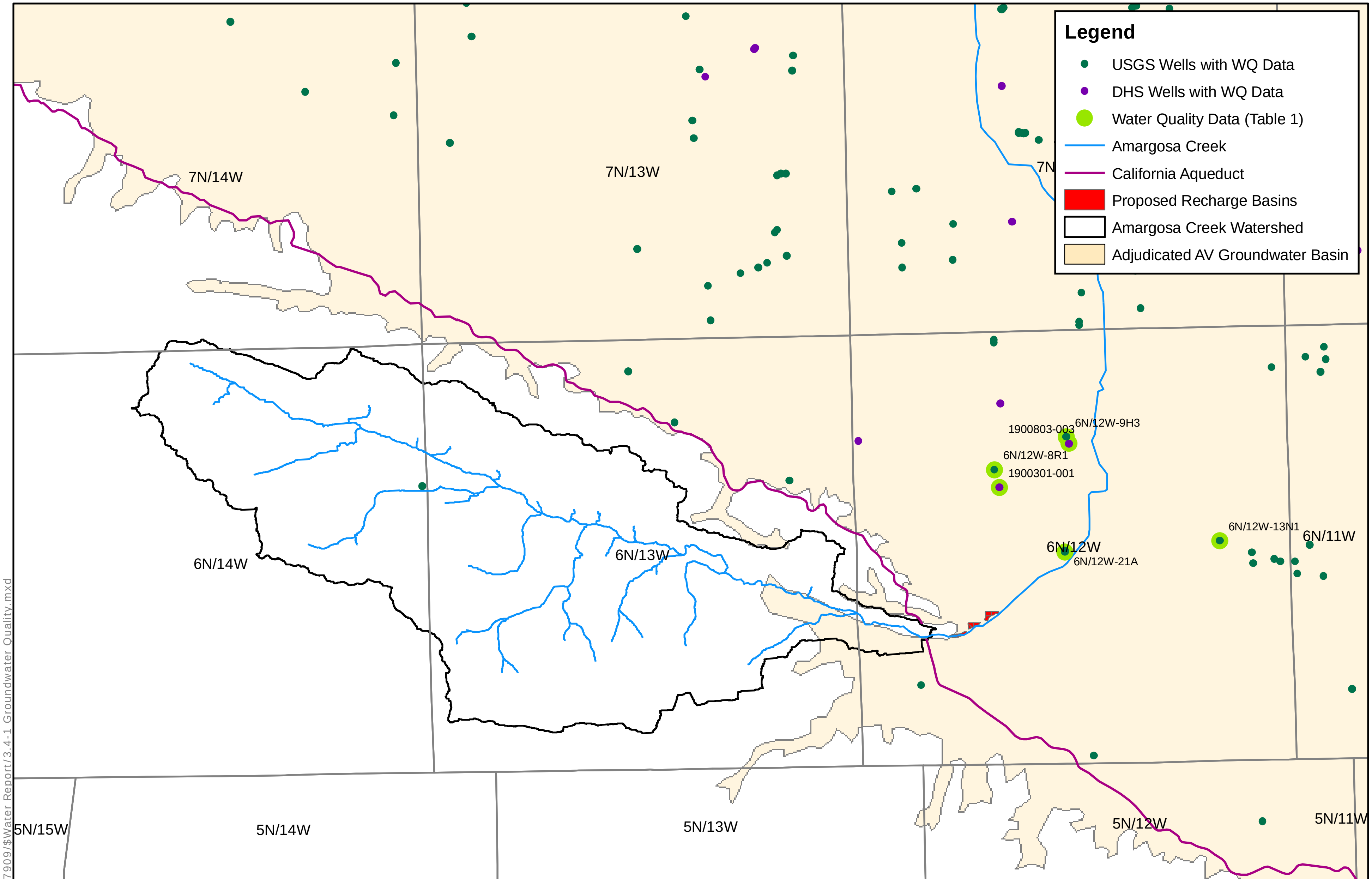
Miles

SAIC
From Science to Solutions

DATE: 10-21-08 BY: A.P.

FIGURE:

3-10



File: Palmdale/TB_WAA_7909/$Water Report/3.4-1 Groundwater Quality.mxd

NOTES:

Coordinate System: GCS North American 1983
Horizontal Datum: NAD 83

Wells with Water Quality Data

0 0.5 1 2 3 4
Miles

SAIC
From Science to Solutions

DATE: 07-21-08

BY: JD

FIGURE:

3-11

Table 3-2: Summary of Groundwater Quality Measurements

Well Number	Sample Date	Total Dissolved Solids (mg/L) (secondary MCL = 500 mg/L)	Nitrates (as NO3) (mg/L) (MCL = 45 mg/L)	Arsenic (mcg/L) (MCL = 10 mcg/L)
USGS				
6N/12W-8R1	1964-03-31	232	1.5	
	1964-07-30	581		3
	1972-09-22	815	3.5	
6N/12W-9H3	1993-09-10			
	1994-09-22	483		
	1995-07-11			
	1996-06-11			
	1997-07-09	521		
	1998-07-28			
	1999-07-15			
	2000-07-27	543		
	2001-07-18			
	2002-07-16	573		
	2002-09-17			2
	2003-08-28			
	2004-08-17			
6N/12W-13N1	1967-06-20	141	0.8	
	1968-06-12	184	0.04	0
	1972-03-15	143	1	
	1972-07-07	175	0.75	
	1973-02-28	129	1.4	
	1974-06-07	193	0	
	1974-06-28	186	3.3	
	1975-04-10	194	0.5	
	1976-04-21	195	0.2	
	1977-05-04	181	1.4	
	1978-05-20	189	0.4	
6N/12W-21A1	1964-05-19	630	6	
	1965-05-20	592	4	
	1965-11-29	734	4	
	1966-06-09	786	6	
	1966-12-13	590	8	
	1967-06-16	627	4	
	1968-06-16	264	4	
	1969-11-19	205	4	
DHS				
1900301-001	2001-02-20	880	17	0
	2001-09-26	980		
	2002-01-15		4.01	
1900803-003	2000-03-02	500	4	0
	2003-07-08	600	5	0
	2005-03-11			
	2005-08-29		6	
	2005-11-21			

4 MODELING

The proposed recharge facility of the UAP increases groundwater supplies to the Antelope Valley Groundwater Basin by augmenting the natural process of recharge to the principal aquifer by capturing streamflow that historically evaporated from lake beds and focusing that water to a point of infiltration up-gradient of historical extractions. This requires careful consideration to the timing of water availability, available storage capacity within the system both above ground and below, as well as the extraction location of water supplies for beneficial use. Facility capacity constraints provide an indication of the maximum amount of water that could be collected for recharge. Operational and scheduling constraints must be in balanced with the availability of water in time and the capacity of the vadose zone to convey these waters and ultimately of the aquifer to store these waters in anticipation of use at a later date. An exact understanding of the availability of water, and the aquifer characteristics and conveyance is not a tractable goal. However, the natural complexity and variability of the system can be simplified in an effort to quantify the hydrology and allow for estimates to be made of the general nature of water availability and disposition. The following sections present analyses of runoff from the Amargosa Creek watershed and recharge to the principal aquifer through the UAP.

4.1 CONCEPTUAL WATERSHED MODEL

The complexity and variability of the geology and hydrologic processes that produce runoff within the Amargosa Creek and in part groundwater recharge of the Lancaster subunit have been simplified and presented in the conceptual model that follows. These simplifications allow for mathematical equations to represent the hydrologic runoff response of the watershed to rainfall, to make estimates of streamflow and recharge, and to support the planning and design of the UAP.

The Antelope Valley groundwater basin is comprised of two aquifers, the unconfined "principal aquifer" and the confined "deep" aquifer. Rainfall less evapotranspiration occurring in the Sierra Pelona Mountains results in runoff collected in the Amargosa Creek with little storage locally in the Natural Watershed. Engineered storm drain systems convey water from the urban landscape to the channel at discrete points along the Amargosa Creek. Channel bed seepage occurs along the length of the Amargosa Creek down-stream from the UAP for approximately ten miles to north of Avenue J where finer silt and clay playa deposits impede seepage and recharge to the principle aquifer. The groundwater contours express a local gradient and flow path from the UAP to the north and east towards the City of Lancaster and Plant 42.

The following assumptions constrain the natural complexity and reduce the need to make observations at every place:

1. All precipitation occurs as rainfall over the Amargosa Creek watershed and is either evapotranspired or conveyed in the channel to the Antelope Valley.
2. The Natural Watershed does not leak (water is not lost to an unknown place).
3. There is no baseflow from the Natural Watershed.
4. Channel bed infiltration and percolation (seepage) to the principal aquifer occurs downstream of the UAP to Avenue J. Downstream of Avenue J finer, less permeable playa deposits impede these processes.
5. Single valued parameters represent the average condition of the system.

4.2 DAILY RUNOFF MODEL (PAIRED WATERSHED)

Amargosa Creek is an ephemeral stream and is dry throughout most the year. Precipitation that occurs on the watershed produces runoff for a short duration. There is no historical long-term gaging station that measured streamflow in the Amargosa Creek. Estimating daily runoff provides an understanding of the frequency, magnitude and duration of the streamflow in Amargosa Creek near the UAP. Presented in this section is the description of a daily runoff paired watershed model that simulates the daily runoff from the Amargosa Creek watersheds from the Little Rock Creek gage supporting the planning of the UAP. The Little Rock Creek period of record (10/1930 to 02/1938; 09/1939 to 09/1977; 10/1978 to 09/1979; and, 01/2002 to 09/2005), limits the daily runoff model to water year 1964-65 to water year 1976-77. All the estimates of streamflow in Amargosa Creek are calculated on a daily-time step, and presented as monthly totals in the following tables.

4.2.1 Base Period Selection

A benchmark or "base period" of hydrology is fundamental to understanding the expectation of runoff from a place, and the base period is determined in part by identifying a sequence of rainfall years where the base period average is equal to the long-term average of record. Three criteria are typically used to select the base period: (1) the base period average annual rainfall should be equal to the long-term average annual rainfall for the study area, (2) the base period should include wet and dry cycles, and (3) the base period end should be as close to the present as possible. The daily runoff paired watershed model is dependent on gaging station records for Little Rock Creek. Therefore an additional criterion was established, (4) the base period should coincide with available Little Rock Creek streamflow data.

Commonly, a rain gage near or within an area of interest is selected to predicate hydrologic analyses. In this case, Station 122 – Leona Valley is optimally located. The cumulative departure from the mean was computed for Station 122, and graphed to determine an appropriate base period for hydrologic analysis. In the graph, the green line represents mean value. The blue line (cumulative departure from the mean) represents the sum of the annual departures through that water year, since the beginning of the period of record. A cumulative departure from the mean represents the accumulation, since the beginning of the period of record, of the differences (departures) in annual total rainfall volume from the mean value for the period of record. Each year's departure is added to or subtracted from the previous year's cumulative total, depending on whether that year's departure was above or below the mean annual rainfall depth. When the slope of the cumulative departure from the mean is negative, the sequence of years is drier than the mean, and conversely when the slope of the cumulative departure from the mean is positive, the sequence of years is wetter than the mean. When the slope between two points on the line is zero, the rainfall for that sequence of years is the same as the mean value of the period of record.

Using the four criteria, a base period from Water Year 1964-65 to 1976-77 was selected. The mean rainfall for this base period is 14.0 inches/year, which is seven percent less than the period of record mean rainfall of 15.0 inches/year.

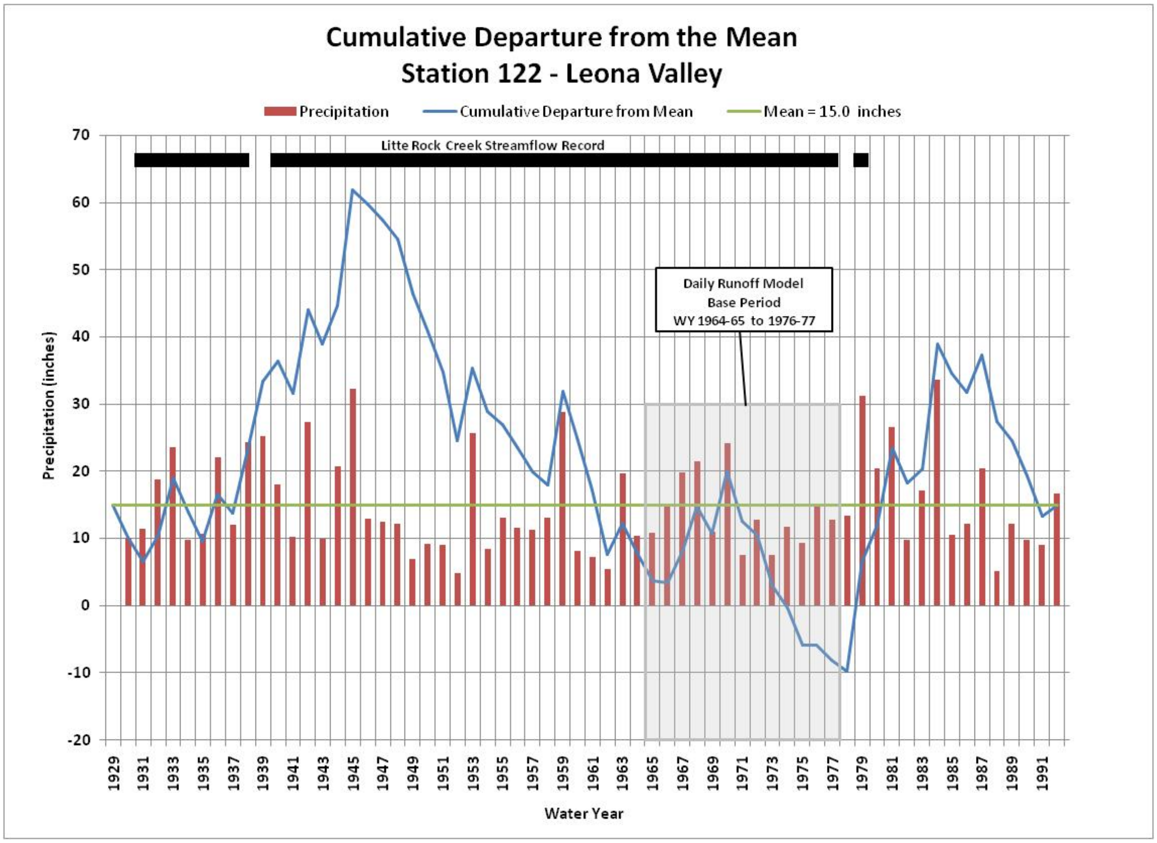


Figure 4-1: Cumulative departure from the mean for Station 122.

4.2.2 Estimates of Amargosa Creek Daily Runoff

Daily runoff from the Natural Watershed is estimated by pro-ration with a paired watershed. The estimated discharge is the measured discharge from the paired watershed times the ratio of watershed areas and precipitation as formalized below (SWRCB 2006):

$$Q_2 = Q_1(A_2/A_1)(I_2/I_1)$$

where:

Q_2 = Daily runoff (cfs) at point of interest on tributary watershed;
Q_1 = Daily runoff (cfs) at nearby gage;
A_2 = Watershed area above point of interest;
A_1 = Watershed area above nearby gage;
I_2 = Precipitation at point of interest; and
I_1 = Precipitation at nearby gage.

The nearest streams to Amargosa Creek with runoff measurements are Little Rock Creek and Big Rock Creek. Little Rock Creek is closer (11 miles to the east) and the gradient is more similar to Amargosa Creek (Table 4-1) and therefore, was selected preferentially to Big Rock Creek because of it is closer (11 miles to the east) and the gradient is more similar to Amargosa Creek (Table 4-1). The ratio of areas (A_2/A_1) is 0.59 (29 sq miles divided by 49 sq. miles).

Table 4-1: Paired Watershed Characteristics

Characteristic	Amargosa Creek	Little Rock Creek	Big Rock Creek
Area (sq miles)	29	49	23
Longest Flow Path Upstream Elevation (feet above sea level, ft asl)	4,218	7,979	8,600
Longest Flow Path Downstream Elevation (ft asl)	2,766	3,280	4,064
Flow length (ft)	76,400	91,080	43,350
Stream Gradient	0.019	0.052	0.105

Station 122 (I_2) was selected for the rainfall input representing the Amargosa Creek Watershed. An area-weighted daily rainfall was estimated for Little Rock Creek watershed (I_1) based on available daily records from Los Angeles County Department of Public Works. The Thiessen Polygon method was used to calculate the area-weighted daily rainfall for the watershed, by subdividing the watershed into the areas covered by each rain gage and area-weighting the daily rainfall for the watershed.

The Little Rock Creek streamflow record (Q_1) was used to predict the streamflow in the Amargosa Creek at the POD (Q_2) scaled by the ratio of area (A_2/A_1) and rainfall (I_2/I_1). However, the historical record of streamflow in Little Rock Creek contains flow most days throughout the year, with base flow through late spring and early summer. Amargosa Creek is an ephemeral stream, with runoff occurring only during periods of intense rainfall (Metzger et al 2002). To account for the lack of base flow in Amargosa Creek, the estimated streamflow in Amargosa Creek is assumed to be zero if the three day running-average of the weighted average rainfall in the Amargosa Creek watershed is less than one-tenth of an inch. The selection of a three-day running average is based on the empirical formula $N = A^{0.2}$, where N is the number of days from the time of the peak to end of the event flow and A is the watershed area in square miles (Dingman 2002). For Amargosa Creek watershed above the point of diversion, N = 1.96 days. Therefore a three-day running average of rainfall accounts for potential runoff from a storm event in the prior two days. The selection of a tenth of an inch for the cutoff for a precipitation event to produce runoff is based on the historic data concerning the streamflow response of Little Rock Creek to precipitation events. In addition, if the area-weighted average rainfall for either Little Rock Creek or Amargosa Creek is less than a tenth of an inch, the rainfall factor (I_2/I_1) is set equal to 1 and does not affect the stream flow for those days.

Over the 13-year period of analysis, the average runoff is estimated to be 2,600 AFY (Table 4-2). The maximum annual runoff for the period of analysis occurred in Water Year 1968-69 and is estimated to be from 10,000 AFY.

Table 4-2: Amargosa Creek Paired Watershed Estimated Monthly Natural Streamflow at POD (in Acre-feet)

	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Water Year
1964									0	0	0	62	
1965	16	0	6	152	0	0	0	0	0	0	4,168	4,045	236
1966	75	254	0	0	0	0	0	0	0	0	147	2,177	8,541
1967	239	0	345	1,106	0	0	0	0	2	0	1,061	120	4,016
1968	45	140	130	51	0	0	0	0	0	0	6	20	1,547
1969	4,979	3,877	347	736	0	0	40	0	0	0	64	0	10,004
1970	11	186	824	0	0	0	0	0	0	0	569	286	1,085
1971	120	61	75	64	9	0	0	0	0	0	6	1,139	1,185
1972	0	0	0	0	0	0	0	0	0	0	16	0	1,145
1973	146	2,042	868	0	0	0	0	0	0	0	21	0	3,071
1974	53	0	347	225	0	0	0	0	0	4	0	71	646
1975	0	79	644	451	0	0	0	0	0	0	0	0	1,249
1976	0	389	179	87	0	0	0	0	218	0	8	3	874
1977	117	27	43	0	214	0	0	0	0				413
Average	**446**	**543**	**293**	**221**	**17**	**0**	**3**	**0**	**16**	**0**	**467**	**610**	**2,616**

Note: Water Year 1965 is defined as the period from Oct 1, 1964 through Sept 30, 1965.

4.2.3 Peak Steam Flow Rate at the Point of Diversion

The 100-year flood event and the 50-year Capital Flood event (LACDPW 2006) for Amargosa Creek near 10th Street have been reported to be 2,350 cfs and 3,695 cfs, respectively (PACE 2003). The Capital Flood flow was derived from 50-year frequency rainfall intensities assuming conditions of saturated soils and post-burned watersheds with a bulking factor that accounts for debris resulting from burned area runoff. The stream reach study presented in the PACE 2003 is located approximately one mile downstream of the project area and includes additional runoff from urban areas below the proposed project site.

The maximum mean daily runoff rate for Amargosa Creek at the proposed point of diversion, derived by pro-rating Little Rock Creek stream flow from Water Year 1965 to 1977 was estimated to be 1,200 cfs in December 1965. The 100-year flood event and the 50-year Capital Flood event estimates are instantaneous peak flows and therefore are higher and not directly comparable the maximum mean daily flow rate.

4.2.4 Proposed Recharge Rate

Two percolation tests at the project site measured preliminary infiltration rates of 3 and 11 feet per day (fpd) (SAIC 2007). The recharge rates of recharge basins are initially high and then decline as recharge progresses, due to surface clogging with fine sediments and biological growth (Fetter 2001) and the differences between the rates infiltration and percolation. For the purpose of this analysis, 5 fpd is used as the recharge rate of the proposed recharge basins near Amargosa Creek. The conceptual design of the Amargosa Creek Recharge Project includes 20 acres of recharge basins. Using a 5 fpd recharge rate and 20 acres of recharge basins, the recharge rate is approximately 100 AFD or approximately 50 cfs of continuous water supply to the basins for each day. Assuming two-thirds of the 20-acre recharge area is on average operational (one-third is dry for maintenance), the average annual recharge volume is 24,000 AF.

4.2.5 Diversion Potential at POD

Recharge basin capacity and the preliminary recharge rate suggest an average daily diversion rate of 50 cfs to supply the recharge basins. The streamflow in Amargosa Creek is flashy and will likely occur over periods of hours, rather than days. A diversion rate capacity of 100 cfs is recommended in order to capture up to 100 AFD on average.

The maximum annual volume that could be potentially diverted by capturing up to 100 AFD on average is 2,800 AFY based on a repeat of the hydrology in water year 1968-1969 (Table 4-3). The average diversion potential for the period of analysis (1965 to 1977) is 1,100 AFY.

Table 4-3: Estimated Diversion Potential at POD with an Average Daily Diversion Rate of 100 AFD (in Acre-feet)

	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Water Year
1964										0	0	62	
1965	16	0	6	152	0	0	0	0	0	0	913	647	236
1966	75	254	0	0	0	0	0	0	0	0	147	496	1,887
1967	239	0	329	1,068	0	0	0	0	2	0	567	120	2,282
1968	45	140	130	51	0	0	0	0	0	0	6	20	1,052
1969	930	987	284	495	0	0	40	0	0	0	64	0	2,762
1970	11	186	484	0	0	0	0	0	0	0	192	286	745
1971	120	61	75	64	9	0	0	0	0	0	6	578	807
1972	0	0	0	0	0	0	0	0	0	0	16	0	583
1973	146	637	790	0	0	0	0	0	0	0	21	0	1,589
1974	53	0	347	201	0	0	0	0	0	4	0	71	622
1975	0	79	601	392	0	0	0	0	0	0	0	0	1,147
1976	0	329	179	87	0	0	0	0	191	0	8	3	787
1977	117	27	43	0	214	0	0	0	0				413
Average	**135**	**208**	**252**	**193**	**17**	**0**	**3**	**0**	**15**	**0**	**149**	**176**	**1,147**

Note: Water Year 1965 is defined as the period from Oct 1, 1964 through Sept 30, 1965.

4.3 CHANNEL SEEPAGE MODEL

The amount of water available for diversion is the water that occurs at the POD and that would not otherwise seep into the channel bed downstream of the POD and recharged the aquifer. Runoff from the Natural Watershed above the POD and from the Urban Watershed from the City of Palmdale together contributes streamflow into Amargosa Creek downstream from the POD. A portion of the streamflow seeps into the channel bed and recharges the aquifer, providing beneficial use; the remaining streamflow enters Lake Lancaster at Avenue H, then Piute Ponds and then Rosamond Dry Lake. This section presents: 1) the urban runoff from the City of Palmdale into Amargosa Creek; 2) the estimated channel seepage for Amargosa Creek downstream of the UAP to Avenue J; and, 3) the proposed diversion that could be diverted without reducing the existing channel seepage between the POD and Avenue J.

4.3.1 Amargosa Creek Streamflow at POD

The POD is located in Amargosa Creek downstream of the Leona Siphon on the California Aqueduct and upstream of the 25th Street Bridge. The watershed above the point of diversion encompasses 29 square miles or approximately 18,600 acres. Estimates of daily streamflow at the POD were developed by pro-rating the gaged streamflow at Little Rock Creek by the area and the rainfall for the period of analysis from 1964-65 to 1976-1977 (Section 4.2). The period of analysis was limited 13 years due to the available data from Little Rock Creek.

4.3.2 Urban Watershed Runoff

Urban Watershed runoff contributes streamflow to Amargosa Creek downstream of the POD. The Urban Watershed within the City of Palmdale downstream of the POD and the City of Lancaster to the detention basin at Avenue H is delineated based on CAD renderings of the City of Palmdale storm drainage network, the City of Lancaster 2005 storm drainage master plan, and the Antelope Valley Integrated Regional Water Management Plan (Kennedy Jenks 2007). The watershed is separated into 25 catchments based on the storm drains and surface features (Figure 4-2). Fifteen of the catchments discharge runoff into Amargosa Creek upstream of Avenue J and contribute to channel seepage. The time of concentration is calculated for each catchment according the Los Angeles County Department of Public Works (LACDPW) 2006 Hydrology Manual using the LACDPW Time of Concentration calculator. The area, proportion of impervious area, soil type, associated 50-yr rainfall intensity isohyet, flow path length and slope are estimated for each catchment for input into the time of concentration calculator. The time of concentration results are found in Table 4-4. The maximum time of concentration is 162 min (2.7 hrs).

For the fifteen catchments that contribute to channel seepage, the Rational Method is used to estimate the runoff:

$$Q = CIA$$

where:

Q = Runoff (L^2/T);
C = Runoff coefficient (unitless);
I = Rainfall intensity (L/T);
A= Area (L^2).

The purpose of rainfall-runoff estimate is to quantify the average daily runoff from each catchment to Amargosa Creek using daily rainfall, and therefore, it is appropriate to apply the rational method to the catchments as long as the maximum time of concentration (2.7 hrs) is not greater than the time step of the rainfall data (24 hrs).

Table 4-4: Catchment Characteristics

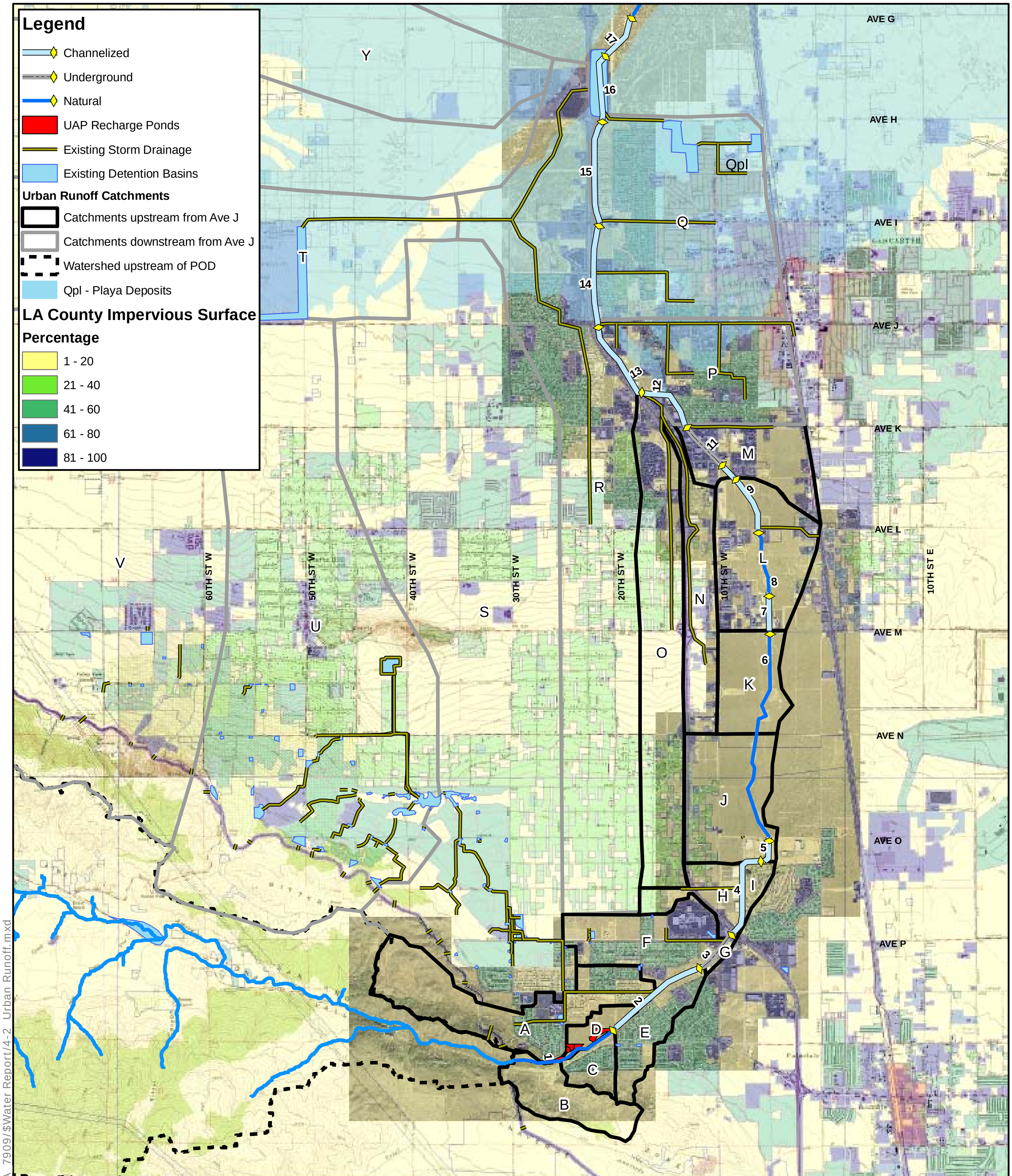
Catchment	Area (acres)	Impervious Portion (%)	Runoff Coefficient	Time of Concentration (min)
A	839	14%	21%	68
B	355	1%	11%	99
C	142	17%	24%	31
D	104	1%	11%	36
E	306	29%	33%	43
F	552	38%	40%	87
G	27	42%	44%	11
H	250	23%	28%	78
I	74	3%	12%	64
J	701	10%	18%	129
K	414	1%	11%	149
L	801	23%	28%	98
M	504	46%	47%	89
N	560	40%	42%	120
O	1,271	25%	30%	162

The runoff coefficient (C) is estimated for each catchment according to the LACDPW 2006 Hydrology Manual, utilizing the area-weighted average of the estimates of impervious area and soil type (Table 4-4). The rainfall intensity (I) is estimated using the historical daily rain gage record for the Palmdale station (1058). The maximum annual volume that flows into the Amargosa Creek from the Urban Watershed is 1800 AFY based on a repeat of the hydrology in water year 1968-1969 (Table 4-5). The average annual volume that flows into the Amargosa Creek from the Urban Watershed is 1100 AFY for the period of analysis (1965 to 1977).

Table 4-5: Estimated Urban Runoff downstream of POD and upstream of Avenue J (in Acre-feet)

	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Water Year
1964									0	0	0	48	
1965	8	0	40	315	0	0	0	0	0	0	1,097	420	412
1966	74	185	0	0	0	0	0	0	0	0	330	332	1,776
1967	269	0	57	368	0	0	0	0	4	0	725	143	1,360
1968	55	74	160	92	0	0	0	0	0	0	13	95	1,248
1969	687	878	11	137	0	0	27	0	0	0	128	0	1,847
1970	8	155	134	0	0	0	0	0	0	0	597	364	427
1971	6	27	61	84	0	0	0	0	0	0	0	893	1,139
1972	0	0	0	0	0	0	0	0	0	0	177	0	893
1973	332	611	303	0	0	0	0	0	0	0	74	0	1,423
1974	717	0	269	0	0	0	0	0	0	158	0	271	1,059
1975	0	149	187	162	0	0	0	0	0	0	0	0	927
1976	0	374	126	13	0	0	0	0	391	0	46	11	904
1977	614	4	84	0	340	0	0	0	0				1,099
Average	**213**	**189**	**110**	**90**	**26**	**0**	**2**	**0**	**28**	**12**	**245**	**198**	**1,116**

Note: Water Year 1965 is defined as the period from Oct 1, 1964 through Sept 30, 1965.



File: Palmdale/Projects/WAA 7909/$Water Report/4-2 Urban Runoff.mxd

NOTES:

Coordinate System: UTM Zone 11N
Horizontal Datum: NAD 83
Aerial Photo 2006 LARIAC
Topo map - USGS 24K Quads
Qpl - Ponit and others 1981
Catchments: A-Y
Creek Sections: 1-19

N

Amargosa Creek
Urban Runoff Catchments

0 0.25 0.5 1 1.5 2

Miles

SAIC From Science to Solutions

FIGURE: 4-2

DATE: 10-03-08

BY: JD

4.3.3 Channel Seepage

Streamflow seeps through the channel bed and percolates downward to recharge the underlying aquifer when the bedload is permeable and the water table is sufficiently deep to not reject the streamflow. The seepage rate is proportion to the product of the wetted area and the infiltration rate of the bedload material:

$$\text{Seepage Rate } [L^3/T] = (\text{Wetted Area } [L^2]) \ (\text{Infiltration Rate } [L/T])$$

The wetted area and bedload infiltration rate varies along the Amargosa Creek channel from the diversion point to Rosamond Dry Lake. Wetted area depends upon the amount of water in the channel and the channel bottom geometry, both of which vary along the channel length. Wetted area is estimated for a given flow rate using the Manning's equation, and requires channel cross-sections, slope, and estimates of the roughness coefficient of the channel bottom (Manning's n). For Amargosa Creek this cross-sectional data downstream of the UAP is not readily available. Nonetheless, this complexity can be reduced by formally relating the wetted area to the flow rate through an empirically derived equation.

4.3.3.1 Wetted Width

An estimate of the wetted area can be made by multiplying the wetted width and the wetted length along a channel.

NATURAL CHANNELS

The wetted width (w) can be estimated from the empirical relationship formalized in the following equation for natural channels (Leopold and others (1964)):

$$w = a_w Q^c,$$

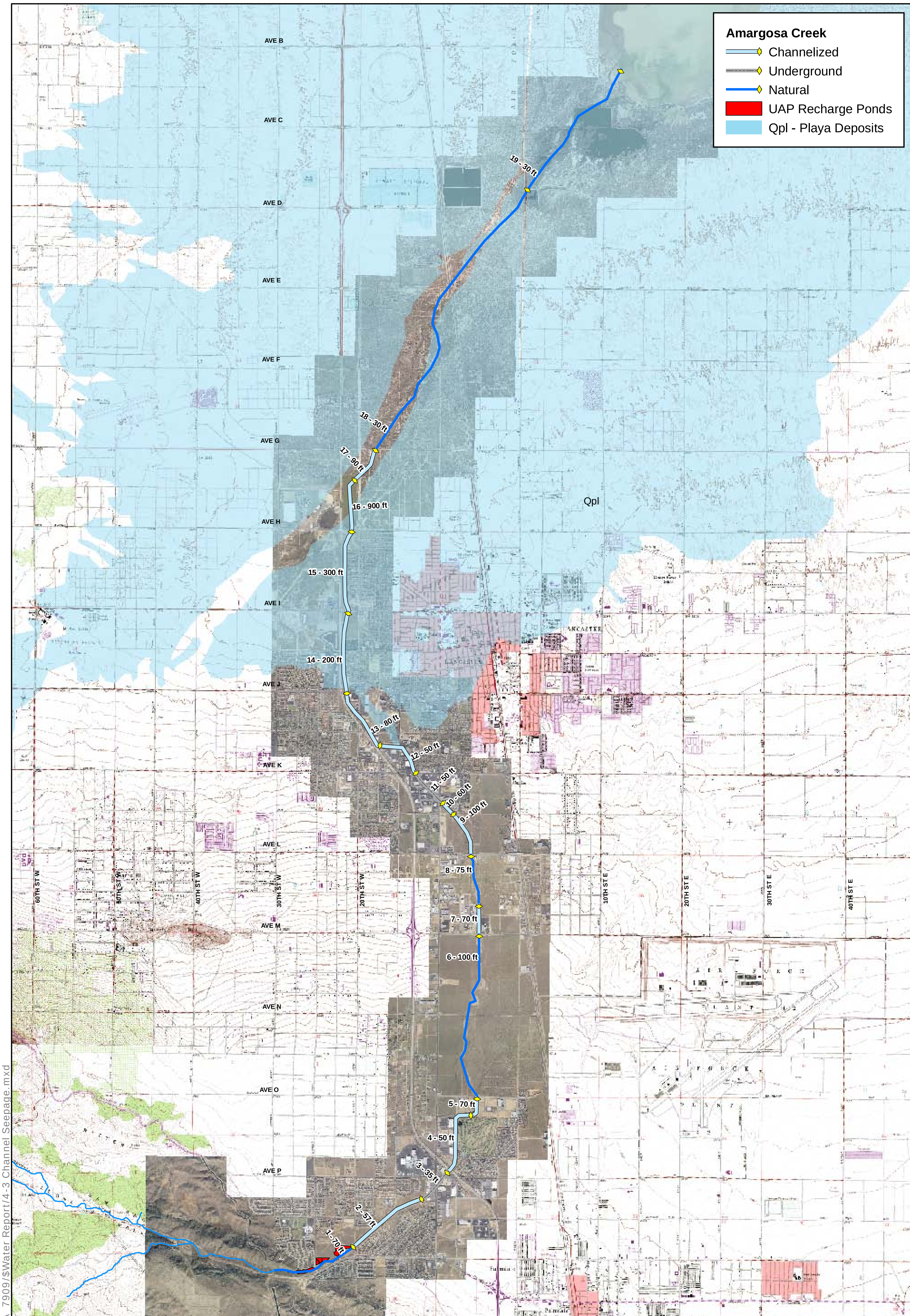
where:

w = Wetted width [L];
Q = Streamflow [L^3/T];
a_w = Coefficient for the relation [T/L^2];
c = Unitless exponent.

The exponent (c) is approximately 0.5 for ephemeral streams within the western regions of the United States, and the coefficient (a_w) is approximately 6 s/ft^2. The wetted width of natural channel was limited to the width of the channel banks, which was obtained from the HEC-RAS data or measured on 2006 aerial photography.

ENGINEERED CHANNELS

For engineered channel sections of the creek, the wetted width is limited to the width of the channel during high water, and is similar to natural conditions during low water. Channel width varies from 50 ft to 300 ft.



File:Palmdale/Projects/WAA_7909/$Water Report/4-3 Channel Seepage.mxd

NOTES:
Coordinate System: UTM Zone 11N
Horizontal Datum: NAD 83
Topo map - USGS 24K
Aerial Photo -2006 LARIAC
Qpl - Ponti and others 1981

1 - 70 ft = Section number - Channel width

Amargosa Creek
Sections for Channel Seepage

0 0.5 1 2 3 4
Miles

From Science to Solutions

FIGURE:

4-3

DATE: 10-3-08

BY: J. Degner

4.3.3.2 Wetted Area

The channel characteristics (natural or channelized, channel width, channel length) were estimated for Amargosa Creek from the UAP to Rosamond Dry Lake bed using HEC-RAS cross-section data or aerial photography where no HEC-RAS data exist. The Amargosa Creek channel was divided into 19 reaches based on similar channel morphologies (Figure 4-4). Utilizing these 19 reach lengths and widths, the wetted area was estimated for the Amargosa Creek for stream flow values (Q) from 0 to 1,400 cfs.

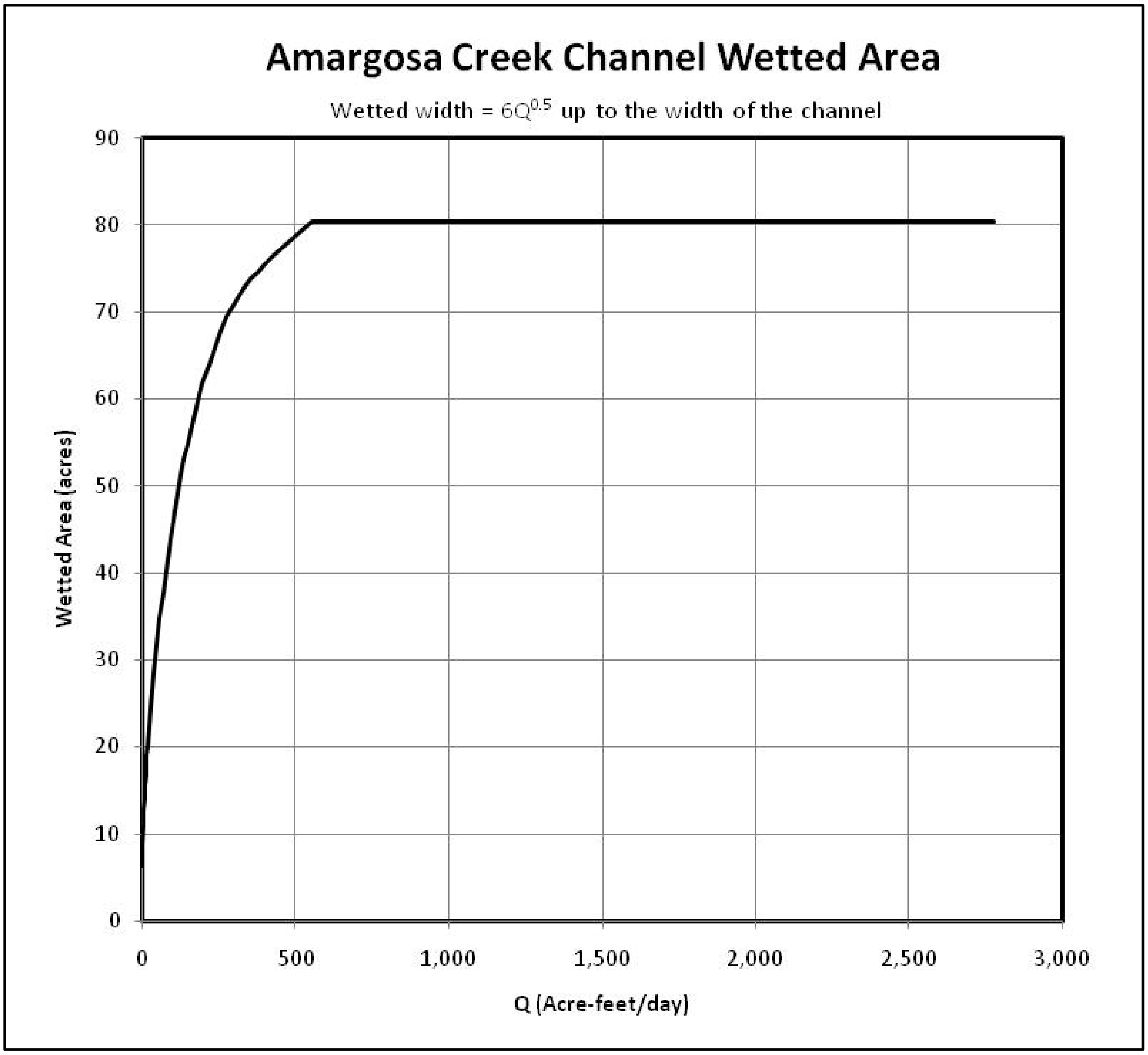


Figure 4-4: Amargosa Creek Channel Wetted Area

4.3.3.3 Infiltration Rate

The infiltration rate is 3.6 ft/day, based on estimates developed for the Problem Statement by the Technical Committee for the Antelope Valley Groundwater Litigation (Appendix C, page 30). The infiltration rate was developed by analyzing limited historic gage data on Big Rock Creek and estimating the channel losses by calculating the difference in the stream gage measurements between an upstream gage in the mountain front and a downstream gage in Big Rock Creek wash. Twenty miles to the east of

Big Rock Creek, pond infiltration tests were performed along the Oro Grande wash, near Victorville, and an average infiltration rate of 3.8 ft/day (Izbicki et al 2007) was determined.

Based on the analysis of the quaternary deposits by Ponti and others (1981) and underlying hydrogeology described by borehole and well logs, increasing fine clays and silts deposited in the playa lakebed occur downstream of Avenue J. These fine materials are less permeable then the fine sands that dominate the Amargosa Creek upstream. Therefore infiltration rate of the streambed downstream of Avenue J (Section 14) is reduced to zero.

4.3.3.4 Channel Seepage Results

The channel seepage was estimated for each of the 19 reaches of the Amargosa Creek from POD to Avenue J for streamflow in that reach. For each reach, the streamflow entering the reach Creek plus the Urban Watershed contributing runoff to that reach minus the channel seepage for that reach is total flow to the next downstream reach. The total channel seepage is the difference between the sum of input streamflows and the streamflow past Avenue J (Table 4-6). The maximum annual channel seepage volume from the POD to Avenue J is 5,700 AFY based on a repeat of the hydrology in water year 1968-1969 (Table 4-6). The average annual channel seepage volume from the POD to Avenue J is 2,200 AFY for the period of analysis (1965 to 1977).

Table 4-6: Estimated Channel Seepage from POD to Avenue J including seepage from Urban Runoff (in Acre-feet)

	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Water Year
1964									0	0	0	107	
1965	25	0	39	402	0	0	0	0	0	0	2,386	1,293	572
1966	130	371	0	0	0	0	0	0	0	0	340	969	4,180
1967	425	0	382	1,356	0	0	0	0	7	0	1,167	237	3,479
1968	87	200	230	125	0	0	0	0	0	0	18	96	2,048
1969	2,343	2,107	347	758	0	0	66	0	0	0	163	0	5,734
1970	19	308	741	0	0	0	0	0	0	0	590	551	1,231
1971	126	85	124	126	9	0	0	0	0	0	6	1,344	1,612
1972	0	0	0	0	0	0	0	0	0	0	148	0	1,350
1973	383	1,335	1,056	0	0	0	0	0	0	0	77	0	2,922
1974	502	0	527	223	0	0	0	0	0	120	0	252	1,330
1975	0	191	774	557	0	0	0	0	0	0	0	0	1,894
1976	0	614	278	100	0	0	0	0	436	0	45	14	1,427
1977	541	31	115	0	427	0	0	0	0	0	0	0	1,172
Average	**352**	**403**	**355**	**281**	**34**	**0**	**5**	**0**	**32**	**9**	**353**	**347**	**2,227**

Note: Water Year 1965 is defined as the period from Oct 1, 1964 through Sept 30, 1965.

4.3.3.5 Sensitivity Analysis

The channel seepage estimate depends upon the infiltration rate. The sensitivity analysis was conducted by halving the infiltration rate to 1.8 ft/day and doubling the infiltration rate to 7.2 ft/day, to understand the sensitivity of channel seepage estimate to infiltration rate. A four-fold change in the infiltration rates results in a three-fold change in channel seepage rates at the larger flows (Figure 4-5).

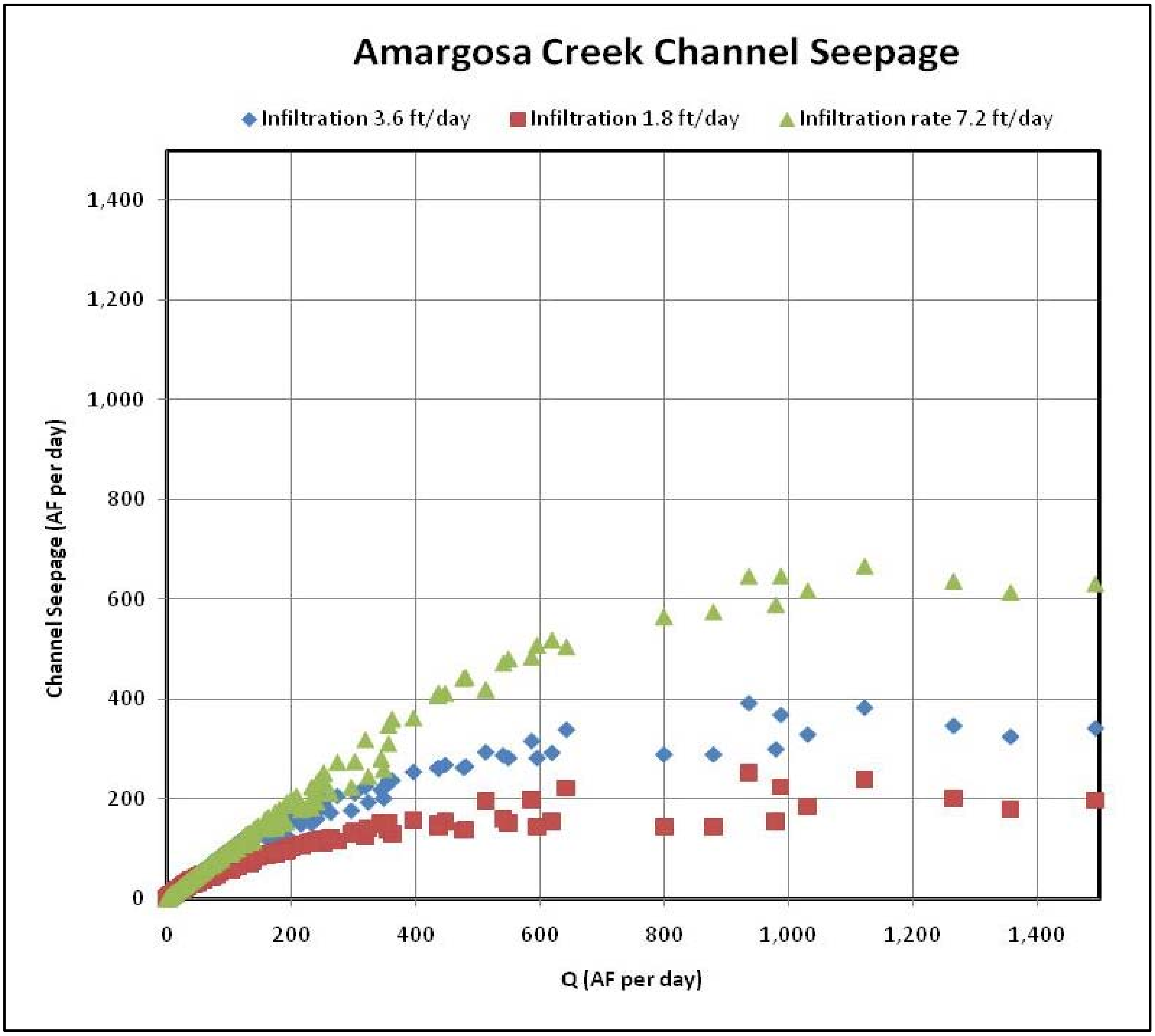


Figure 4-5. Amargosa Creek Channel Seepage

4.3.4 Streamflow at Avenue J

As presented in the channel seepage section, channel seepage is assumed to occur from the POD to Avenue J. The amount of runoff at Avenue J is the natural runoff at POD (Table 4-2) plus urban runoff (Table 4-5) minus the channel seepage (Table 4-6). The maximum annual volume that flows past Avenue J is 6,100 AFY based on a repeat of the hydrology in water year 1968-1969 (Table 4-7). The average annual volume that flows past Avenue J is 1,500 AFY for the period of analysis (1965 to 1977).

Table 4-7: Total Streamflow at Avenue J assuming infiltration rate of 3.6 ft/day (in Acre-feet)

	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Water Year
1964										0	0	4	
1965	0	0	7	65	0	0	0	0	0	0	2,878	3,172	76
1966	18	68	0	0	0	0	0	0	0	0	137	1,540	6,136
1967	83	0	19	118	0	0	0	0	0	0	619	26	1,897
1968	12	13	60	18	0	0	0	0	0	0	0	19	747
1969	3,323	2,649	10	114	0	0	2	0	0	0	30	0	6,117
1970	0	34	217	0	0	0	0	0	0	0	576	99	280
1971	0	3	12	21	0	0	0	0	0	0	0	688	712
1972	0	0	0	0	0	0	0	0	0	0	44	0	688
1973	95	1,318	115	0	0	0	0	0	0	0	17	0	1,571
1974	267	0	89	2	0	0	0	0	0	42	0	90	375
1975	0	37	57	56	0	0	0	0	0	0	0	0	282
1976	0	149	28	0	0	0	0	0	173	0	10	0	350
1977	190	0	12	0	128	0	0	0	0				340
Average	**307**	**329**	**48**	**30**	**10**	**0**	**0**	**0**	**13**	**3**	**332**	**434**	**1,506**

Note: Water Year 1965 is defined as the period from Oct 1, 1964 through Sept 30, 1965.

4.3.5 Diversion at POD based on Streamflow at Avenue J

The diversion at POD based on streamflow at Avenue J is the volume that could be diverted without reducing the existing channel seepage between the POD and Avenue J. This proposed diversion is estimated for each day in the period of analysis as the minimum of three estimates: the streamflow at the POD; the estimated total runoff at Avenue J; and the diversion capacity of 100 AFD (Table 4-3). The maximum annual proposed diversion at POD based on streamflow at Avenue J is 1,400 AFY based on a repeat of the hydrology in water year 1968-1969 (Table 4-8). The average annual proposed diversion at POD based on streamflow at Avenue J is 400 AFY for the period of analysis (1965 to 1977).

Table 4-8: Diversion at POD based on Streamflow at Avenue J (in Acre-feet)

	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Water Year
1964										0	0	2	
1965	0	0	6	56	0	0	0	0	0	0	682	291	65
1966	18	68	0	0	0	0	0	0	0	0	86	222	1,059
1967	46	0	19	118	0	0	0	0	0	0	307	26	492
1968	12	13	57	12	0	0	0	0	0	0	0	8	427
1969	706	593	10	114	0	0	2	0	0	0	30	0	1,433
1970	0	34	139	0	0	0	0	0	0	0	127	24	203
1971	0	3	12	21	0	0	0	0	0	0	0	386	187
1972	0	0	0	0	0	0	0	0	0	0	2	0	386
1973	54	306	115	0	0	0	0	0	0	0	7	0	476
1974	26	0	58	2	0	0	0	0	0	3	0	36	93
1975	0	11	57	50	0	0	0	0	0	0	0	0	157
1976	0	87	28	0	0	0	0	0	32	0	3	0	148
1977	51	0	9	0	70	0	0	0	0				133
Average	**70**	**86**	**39**	**29**	**5**	**0**	**0**	**0**	**2**	**0**	**96**	**77**	**405**

Note: Water Year 1965 is defined as the period from Oct 1, 1964 through Sept 30, 1965.

The resulting streamflow at Avenue J after diversion at POD is estimated by subtracting the diversion at POD based on streamflow at Avenue J (Table 4-8) from the estimated streamflow at Avenue J on a daily time-step (Table 4-7). The maximum annual remaining streamflow at Avenue J is 4,700 AFY based on a repeat of the hydrology in water year 1968-1969 (Table 4-9). The average annual remaining streamflow at Avenue J is 1,100 AFY for the period of analysis (1965 to 1977).

Table 4-9: Streamflow at Avenue J after Diversion at POD (in Acre-feet)

	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Water Year
1964										0	0	2	
1965	0	0	1	9	0	0	0	0	0	0	2,197	2,881	11
1966	0	0	0	0	0	0	0	0	0	0	50	1,318	5,077
1967	37	0	0	0	0	0	0	0	0	0	312	0	1,405
1968	0	0	2	5	0	0	0	0	0	0	0	11	320
1969	2,617	2,056	0	0	0	0	0	0	0	0	0	0	4,684
1970	0	0	78	0	0	0	0	0	0	0	449	75	78
1971	0	0	0	0	0	0	0	0	0	0	0	302	524
1972	0	0	0	0	0	0	0	0	0	0	42	0	302
1973	41	1,012	0	0	0	0	0	0	0	0	10	0	1,095
1974	241	0	31	0	0	0	0	0	0	39	0	54	282
1975	0	26	0	6	0	0	0	0	0	0	0	0	126
1976	0	62	0	0	0	0	0	0	141	0	7	0	202
1977	139	0	3	0	58	0	0	0	0				207
Average	**237**	**243**	**9**	**2**	**4**	**0**	**0**	**0**	**11**	**3**	**236**	**357**	**1,101**

Note: Water Year 1965 is defined as the period from Oct 1, 1964 through Sept 30, 1965.

4.4 GROUNDWATER MOUNDING

Water delivered to the UAP recharge facility will be retained in basins and will percolate downward to the underlying aquifer. This process will create a mound of water below the basins that will dissipate and move "down gradient" from the basins. Excessive groundwater mounding under the recharge basins may result in groundwater levels beneath the facility to approach the ground surface. Localities that are most susceptible to liquefaction-induced damages are underlain by loose, granular, water saturated sediment within 40 feet of the ground surface (CGS 2002). Groundwater within 40 feet of the surface significantly increases the risk for liquefaction.

Based on water contours presented in the 2008 Problem Statement, the depth to water near the UAP is in the range of 200 ft below ground surface (bgs). Liquefaction is typically not a concern provided the depth to water is 40 ft bgs or greater (CGS 2002). Therefore, UAP recharge has the capacity to raise the water table beneath the facility an additional 150 ft and while providing the 40 ft bgs of unsaturated zone necessary for the liquefaction safety factor.

Monitoring wells will be installed inside and outside the UAP area to ensure the water table remains deeper than the California Geological Society guidelines for liquefaction risk.

4.5 RESULTS

All the stream flow values for Amargosa Creek are computed for a period of analysis from Water Year 1964-65 to Water Year 1976-77 on a daily timestep which are summed to the month (Table 4-10). A description of the average annual results is provided below.

The average annual Amargosa Creek streamflow at the POD is estimated to be 2,600 AFY (section 4.2.2). Downstream of POD to Avenue J, urban runoff contributes an estimated 1,100 AFY on average to Amargosa Creek streamflow (section 4.3.2). Of the combined flows (3,700 AFY), 2,200 AFY is estimated to seep into the channel bed between the POD and Avenue J and provides recharge to the aquifer (section 4.3.3), and 1,500 AFY is estimated to flow past Avenue J and eventually flow into Lake Lancaster at Avenue H, Piute Ponds or Rosamond Dry Lake where recharge is limited due to the finer sediments of the historical and existing lakebeds (section 4.3.4).

The recharge capacity of the proposed spreading basins limits the daily diversion to approximately 100 AF per day. The discharge from Amargosa Creek watershed is flashy and will likely occur over periods of hours, rather than days. An instantaneous diversion rate of 100 cfs is recommended in order to capture 100 AFD over 12 hours of streamflow.

The diversion potential, which is the maximum diversion that is possible from the streamflow at the POD, is 1,100 AFY on average (section 4.2.5). The diversion at POD based on streamflow at Avenue J is the volume that could be diverted without reducing the existing channel seepage between the POD and Avenue J. and is estimated to be 400 AFY (section 4.3.5). Total runoff at Avenue J after the proposed diversion is 1,100 AF on average (section 4.3.5).

Table 4-10: Summary of the Results (in Acre-feet)

Year		Volumes	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Water Year
		Streamflow at POD										0	0	62	
	Current	Urban runoff POD to Ave J										0	0	48	
		Channel Seepage POD to Ave J										0	0	107	
1964		Total Streamflow at Ave J										0	0	4	
		Diversion Potential at POD										0	0	62	
	Proposed	Diversion based on Streamflow at Ave J										0	0	2	
		Streamflow after Diversion at Ave J										0	0	2	
		Streamflow at POD	16	0	6	152	0	0	0	0	0	0	4,168	4,045	**236**
	Current	Urban runoff POD to Ave J	8	0	40	315	0	0	0	0	0	0	1,097	420	**412**
		Channel Seepage POD to Ave J	25	0	39	402	0	0	0	0	0	0	2,386	1,293	**572**
1965		Total Streamflow at Ave J	0	0	7	65	0	0	0	0	0	0	2,878	3,172	**76**
		Diversion Potential at POD	16	0	6	152	0	0	0	0	0	0	913	647	**236**
	Proposed	Diversion based on Streamflow at Ave J	0	0	6	56	0	0	0	0	0	0	682	291	**65**
		Streamflow after Diversion at Ave J	0	0	1	9	0	0	0	0	0	0	2,197	2,881	**11**
		Streamflow at POD	75	254	0	0	0	0	0	0	0	0	147	2,177	**8,541**
	Current	Urban runoff POD to Ave J	74	185	0	0	0	0	0	0	0	0	330	332	**1,776**
		Channel Seepage POD to Ave J	130	371	0	0	0	0	0	0	0	0	340	969	**4,180**
1966		Total Streamflow at Ave J	18	68	0	0	0	0	0	0	0	0	137	1,540	**6,136**
		Diversion Potential at POD	75	254	0	0	0	0	0	0	0	0	147	496	**1,887**
	Proposed	Diversion based on Streamflow at Ave J	18	68	0	0	0	0	0	0	0	0	86	222	**1,059**
		Streamflow after Diversion at Ave J	0	0	0	0	0	0	0	0	0	0	50	1,318	**5,077**
		Streamflow at POD	239	0	345	1,106	0	0	0	0	2	0	1,061	120	**4,016**
	Current	Urban runoff POD to Ave J	269	0	57	368	0	0	0	0	4	0	725	143	**1,360**
		Channel Seepage POD to Ave J	425	0	382	1,356	0	0	0	0	7	0	1,167	237	**3,479**
1967		Total Streamflow at Ave J	83	0	19	118	0	0	0	0	0	0	619	26	**1,897**
		Diversion Potential at POD	239	0	329	1,068	0	0	0	0	2	0	567	120	**2,282**
	Proposed	Diversion based on Streamflow at Ave J	46	0	19	118	0	0	0	0	0	0	307	26	**492**
		Streamflow after Diversion at Ave J	37	0	0	0	0	0	0	0	0	0	312	0	**1,405**
		Streamflow at POD	45	140	130	51	0	0	0	0	0	0	6	20	**1,547**
	Current	Urban runoff POD to Ave J	55	74	160	92	0	0	0	0	0	0	13	95	**1,248**
		Channel Seepage POD to Ave J	87	200	230	125	0	0	0	0	0	0	18	96	**2,048**
1968		Total Streamflow at Ave J	12	13	60	18	0	0	0	0	0	0	0	19	**747**
		Diversion Potential at POD	45	140	130	51	0	0	0	0	0	0	6	20	**1,052**
	Proposed	Diversion based on Streamflow at Ave J	12	13	57	12	0	0	0	0	0	0	0	8	**427**
		Streamflow after Diversion at Ave J	0	0	2	5	0	0	0	0	0	0	0	11	**320**
		Streamflow at POD	4,979	3,877	347	736	0	0	40	0	0	0	64	0	**10,004**
	Current	Urban runoff POD to Ave J	687	878	11	137	0	0	27	0	0	0	128	0	**1,847**
		Channel Seepage POD to Ave J	2,343	2,107	347	758	0	0	66	0	0	0	163	0	**5,734**
1969		Total Streamflow at Ave J	3,323	2,649	10	114	0	0	2	0	0	0	30	0	**6,117**
		Diversion Potential at POD	930	987	284	495	0	0	40	0	0	0	64	0	**2,762**
	Proposed	Diversion based on Streamflow at Ave J	706	593	10	114	0	0	2	0	0	0	30	0	**1,433**
		Streamflow after Diversion at Ave J	2,617	2,056	0	0	0	0	0	0	0	0	0	0	**4,684**
		Streamflow at POD	11	186	824	0	0	0	0	0	0	0	569	286	**1,085**
	Current	Urban runoff POD to Ave J	8	155	134	0	0	0	0	0	0	0	597	364	**427**
		Channel Seepage POD to Ave J	19	308	741	0	0	0	0	0	0	0	590	551	**1,231**
1970		Total Streamflow at Ave J	0	34	217	0	0	0	0	0	0	0	576	99	**280**
		Diversion Potential at POD	11	186	484	0	0	0	0	0	0	0	192	286	**745**
	Proposed	Diversion based on Streamflow at Ave J	0	34	139	0	0	0	0	0	0	0	127	24	**203**
		Streamflow after Diversion at Ave J	0	0	78	0	0	0	0	0	0	0	449	75	**78**
		Streamflow at POD	120	61	75	64	9	0	0	0	0	0	6	1,139	**1,185**
	Current	Urban runoff POD to Ave J	6	27	61	84	0	0	0	0	0	0	0	893	**1,139**
		Channel Seepage POD to Ave J	126	85	124	126	9	0	0	0	0	0	6	1,344	**1,612**
1971		Total Streamflow at Ave J	0	3	12	21	0	0	0	0	0	0	0	688	**712**
		Diversion Potential at POD	120	61	75	64	9	0	0	0	0	0	6	578	**807**
	Proposed	Diversion based on Streamflow at Ave J	0	3	12	21	0	0	0	0	0	0	0	386	**187**
		Streamflow after Diversion at Ave J	0	0	0	0	0	0	0	0	0	0	0	302	**524**
		Streamflow at POD	0	0	0	0	0	0	0	0	0	0	16	0	**1,145**
	Current	Urban runoff POD to Ave J	0	0	0	0	0	0	0	0	0	0	177	0	**893**
		Channel Seepage POD to Ave J	0	0	0	0	0	0	0	0	0	0	148	0	**1,350**
1972		Total Streamflow at Ave J	0	0	0	0	0	0	0	0	0	0	44	0	**688**
		Diversion Potential at POD	0	0	0	0	0	0	0	0	0	0	16	0	**583**
	Proposed	Diversion based on Streamflow at Ave J	0	0	0	0	0	0	0	0	0	0	2	0	**386**
		Streamflow after Diversion at Ave J	0	0	0	0	0	0	0	0	0	0	42	0	**302**

Table 4.10: Summary of Results (continued)

Year		Volumes	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Water Year
1973	Current	Streamflow at POD	146	2,042	868	0	0	0	0	0	0	0	21	0	**3,071**
		Urban runoff POD to Ave J	332	611	303	0	0	0	0	0	0	0	74	0	**1,423**
		Channel Seepage POD to Ave J	383	1,335	1,056	0	0	0	0	0	0	0	77	0	**2,922**
		Total Streamflow at Ave J	95	1,318	115	0	0	0	0	0	0	0	17	0	**1,571**
	Proposed	Diversion Potential at POD	146	637	790	0	0	0	0	0	0	0	21	0	**1,589**
		Diversion based on Streamflow at Ave J	54	306	115	0	0	0	0	0	0	0	7	0	**476**
		Streamflow after Diversion at Ave J	41	1,012	0	0	0	0	0	0	0	0	10	0	**1,095**
1974	Current	Streamflow at POD	53	0	347	225	0	0	0	0	0	4	0	71	**646**
		Urban runoff POD to Ave J	717	0	269	0	0	0	0	0	0	158	0	271	**1,059**
		Channel Seepage POD to Ave J	502	0	527	223	0	0	0	0	0	120	0	252	**1,330**
		Total Streamflow at Ave J	267	0	89	2	0	0	0	0	0	42	0	90	**375**
	Proposed	Diversion Potential at POD	53	0	347	201	0	0	0	0	0	4	0	71	**622**
		Diversion based on Streamflow at Ave J	26	0	58	2	0	0	0	0	0	3	0	36	**93**
		Streamflow after Diversion at Ave J	241	0	31	0	0	0	0	0	0	39	0	54	**282**
1975	Current	Streamflow at POD	0	79	644	451	0	0	0	0	0	0	0	0	**1,249**
		Urban runoff POD to Ave J	0	149	187	162	0	0	0	0	0	0	0	0	**927**
		Channel Seepage POD to Ave J	0	191	774	557	0	0	0	0	0	0	0	0	**1,894**
		Total Streamflow at Ave J	0	37	57	56	0	0	0	0	0	0	0	0	**282**
	Proposed	Diversion Potential at POD	0	79	601	392	0	0	0	0	0	0	0	0	**1,147**
		Diversion based on Streamflow at Ave J	0	11	57	50	0	0	0	0	0	0	0	0	**157**
		Streamflow after Diversion at Ave J	0	26	0	6	0	0	0	0	0	0	0	0	**126**
1976	Current	Streamflow at POD	0	389	179	87	0	0	0	0	218	0	8	3	**874**
		Urban runoff POD to Ave J	0	374	126	13	0	0	0	0	391	0	46	11	**904**
		Channel Seepage POD to Ave J	0	614	278	100	0	0	0	0	436	0	45	14	**1,427**
		Total Streamflow at Ave J	0	149	28	0	0	0	0	0	173	0	10	0	**350**
	Proposed	Diversion Potential at POD	0	329	179	87	0	0	0	0	191	0	8	3	**787**
		Diversion based on Streamflow at Ave J	0	87	28	0	0	0	0	0	32	0	3	0	**148**
		Streamflow after Diversion at Ave J	0	62	0	0	0	0	0	0	141	0	7	0	**202**
1977	Current	Streamflow at POD	117	27	43	0	214	0	0	0	0				**413**
		Urban runoff POD to Ave J	614	4	84	0	340	0	0	0	0				**1,099**
		Channel Seepage POD to Ave J	541	31	115	0	427	0	0	0	0				**1,172**
		Total Streamflow at Ave J	190	0	12	0	128	0	0	0	0				**340**
	Proposed	Diversion Potential at POD	117	27	43	0	214	0	0	0	0				**413**
		Diversion based on Streamflow at Ave J	51	0	9	0	70	0	0	0	0				**133**
		Streamflow after Diversion at Ave J	139	0	3	0	58	0	0	0	0				**207**
Average	Current	Streamflow at POD	446	543	293	221	17	0	3	0	17	0	505	655	**2,616**
		Urban runoff POD to Ave J	213	189	110	90	26	0	2	0	30	13	265	211	**1,116**
		Channel Seepage POD to Ave J	352	403	355	281	34	0	5	0	34	10	412	396	**2,227**
		Total Streamflow at Ave J	307	329	48	30	10	0	0	0	13	3	359	470	**1,506**
	Proposed	Diversion Potential at POD	135	208	252	193	17	0	3	0	15	0	162	185	**1,147**
		Diversion based on Streamflow at Ave J	70	86	39	29	5	0	0	0	2	0	104	83	**405**
		Streamflow after Diversion at Ave J	237	243	9	2	4	0	0	0	11	3	256	387	**1,101**
Maximum (WY 1969)	Current	Streamflow at POD	4,979	3,877	347	736	0	0	40	0	0	0	64	0	**10,004**
		Urban runoff POD to Ave J	687	878	11	137	0	0	27	0	0	0	128	0	**1,847**
		Channel Seepage POD to Ave J	2,343	2,107	347	758	0	0	66	0	0	0	163	0	**5,734**
		Total Streamflow at Ave J	3,323	2,649	10	114	0	0	2	0	0	0	30	0	**6,117**
	Proposed	Diversion Potential at POD	930	987	284	495	0	0	40	0	0	0	64	0	**2,762**
		Diversion based on Streamflow at Ave J	706	593	10	114	0	0	2	0	0	0	30	0	**1,433**
		Streamflow after Diversion at Ave J	2,617	2,056	0	0	0	0	0	0	0	0	0	0	**4,684**
Cumulative	Current	Streamflow at POD	5,799	7,054	3,807	2,871	224	0	40	0	221	4	6,066	7,925	**34,010**
		Urban runoff POD to Ave J	2,769	2,459	1,431	1,170	340	0	27	0	395	158	3,186	2,576	**14,511**
		Channel Seepage POD to Ave J	4,581	5,242	4,613	3,647	436	0	66	0	443	120	4,941	4,863	**28,950**
		Total Streamflow at Ave J	3,988	4,271	625	394	128	0	2	0	173	42	4,310	5,638	**19,572**
	Proposed	Diversion Potential at POD	1,751	2,700	3,270	2,510	224	0	40	0	194	4	1,938	2,284	**14,913**
		Diversion based on Streamflow at Ave J	914	1,114	510	374	70	0	2	0	32	3	1,243	996	**5,259**
		Streamflow after Diversion at Ave J	3,075	3,156	115	20	58	0	0	0	141	39	3,067	4,642	**14,313**

Estimated Volume	Table
Streamflow at POD	4-2
Urban runoff POD to Ave J	4-5
Channel Seepage POD to Ave J	4-6
Total Streamflow at Ave J	4-7
Diversion Potential at POD	4-3
Diversion based on Streamflow at Ave J	4-8
Streamflow after Diversion at Ave J	4-9

5 CONCLUSION

The Antelope Valley Groundwater Basin is comprised of two aquifers, the unconfined “principal aquifer” and the confined “deep” aquifer (section 2.4). Recent groundwater contours express a local gradient and flow path from the UAP to the north and east towards the City of Lancaster and Plant 42 (section 3.3).

Amargosa Creek is tributary to Lake Lancaster (detention basin north of Avenue H), Piute Ponds, and then Rosamond Dry Lake. The Amargosa Creek watershed area upstream of the POD is 29 square miles, which is approximately 20 percent of the watershed area of Lake Lancaster (160 square miles) and approximately 2 percent of the watershed area of Rosamond Dry Lake (1,200 square miles). Engineered storm drain systems convey water from the urban landscape to the channel at discrete points along the Amargosa Creek downstream from the UAP. Channel bed seepage occurs along the length of the Amargosa Creek down-stream from the UAP for approximately ten miles to north of Avenue J where finer silt and clay playa deposits impede seepage and recharge to the principal aquifer (section 2.4). Channel seepage results in recharge to the groundwater.

The recharge capacity of the proposed spreading basins is approximately 100 AF per day, and therefore the daily diversion capacity is limited to 100 AF. The discharge from Amargosa Creek watershed is flashy and will likely occur over periods of hours, rather than days. An instantaneous diversion rate of 100 cfs is recommended in order to capture up to 100 AFD.

Rainfall less evapotranspiration occurring in the Sierra Pelona Mountains results in runoff collected in the Amargosa Creek with little storage locally in the Natural Watershed (section 2.2). For the Amargosa Creek watershed, daily rainfall on average exceeds 1 inch on six days each year in the mountains and 2 days each year in the valley. In the mountains rainfall is expected to exceed 0.2 inches each hour 23 hours each year and 0.5 inches per hour 2 hours each year (section 3.1).

The average annual Amargosa Creek streamflow at the POD is estimated to be 2,600 AFY (section 4.2.2). Downstream of POD to Avenue J, urban runoff contributes an estimated 1,100 AFY on average to Amargosa Creek streamflow (section 4.3.2). Of the combined flows (3,700 AFY), 2,200 AFY is estimated to seep into the channel bed between the POD and Avenue J and provides recharge to the aquifer (section 4.3.3), and 1,500 AFY is estimated to flow past Avenue J and eventually flow into Lake Lancaster at Avenue H, Piute Ponds or Rosamond Dry Lake where recharge is limited due to the finer sediments of the historical and existing lakebeds (section 4.3.4).

The diversion potential, which is the maximum diversion that is possible from the streamflow at the POD, is 1,100 AFY on average (section 4.2.5). The diversion at POD based on streamflow at Avenue J is the volume that could be diverted without reducing the existing channel seepage between the POD and Avenue J. and is estimated to be 400 AFY (section 4.3.5). Total runoff at Avenue J after the proposed diversion is 1,100 AF on average (section 4.3.5).

The effect the diversion would have on the seasonally flooded areas downstream of Lake Lancaster and the seasonal flooding of Rosamond Dry Lake is minimal. The Amargosa Creek watershed above the POD is approximately 2% of the contributing watershed area of Rosamond Dry Lake. Due to the limited recharge capacity at the UAP of 100 AFD and to maintain the existing channel seepage, approximately 80% of the all the streamflow would pass by the point of diversion. Therefore the reduction in volume of seasonal flooding at Rosamond Dry Lake due to the diversion at the POD is approximately 1 percent.

Table 5-1: Summary of Results (all values in Acre-feet per Year)

Year		Volumes	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Water Year
Average	Current	Streamflow at POD	446	543	293	221	17	0	3	0	17	0	505	655	**2,616**
		Urban runoff POD to Ave J	213	189	110	90	26	0	2	0	30	13	265	211	**1,116**
		Channel Seepage POD to Ave J	352	403	355	281	34	0	5	0	34	10	412	396	**2,227**
		Total Streamflow at Ave J	307	329	48	30	10	0	0	0	13	3	359	470	**1,506**
	Proposed	Diversion Potential at POD	135	208	252	193	17	0	3	0	15	0	162	185	**1,147**
		Diversion based on Streamflow at Ave J	70	86	39	29	5	0	0	0	2	0	104	83	**405**
		Streamflow after Diversion at Ave J	237	243	9	2	4	0	0	0	11	3	256	387	**1,101**
Maximum (WY 1969)	Current	Streamflow at POD	4,979	3,877	347	736	0	0	40	0	0	0	64	0	**10,004**
		Urban runoff POD to Ave J	687	878	11	137	0	0	27	0	0	0	128	0	**1,847**
		Channel Seepage POD to Ave J	2,343	2,107	347	758	0	0	66	0	0	0	163	0	**5,734**
		Total Streamflow at Ave J	3,323	2,649	10	114	0	0	2	0	0	0	30	0	**6,117**
	Proposed	Diversion Potential at POD	930	987	284	495	0	0	40	0	0	0	64	0	**2,762**
		Diversion based on Streamflow at Ave J	706	593	10	114	0	0	2	0	0	0	30	0	**1,433**
		Streamflow after Diversion at Ave J	2,617	2,056	0	0	0	0	0	0	0	0	0	0	**4,684**

Estimated Volume	Table
Streamflow at POD	4-2
Urban runoff POD to Ave J	4-5
Channel Seepage POD to Ave J	4-6
Total Streamflow at Ave J	4-7
Diversion Potential at POD	4-3
Diversion based on Streamflow at Ave J	4-8
Streamflow after Diversion at Ave J	4-9

The recharge operations will create a groundwater mound below the recharge basins that will dissipate and move down gradient from the basins to the north and east toward the City of Lancaster and Plant 42 (section 4.4).

The following limitations to the findings are due to lack of data or limited access to data.

- Amargosa Creek streamflow is not gaged and in this report is estimated using the best available data. Gaging stations in Amargosa Creek would provide more accurate estimates of flow.
- The channel seepage estimates are based on reported values not measured values.
- The amount of Amargosa Creek water which is retained in, flows through, evaporates, and percolates to recharge the groundwater at Lake Lancaster was not available. Based on the available boring and geologic mapping, and the persistent ponding of water in Lake Lancaster through the summer in wet years, the percolation is probably negligible.
- Limited data was available for the storm drainage system for most of the City of Lancaster; therefore the urban runoff from most of the City of Lancaster (north of Ave J) into Lake Lancaster was not estimated.
- The sediment flux from the Amargosa Creek watershed upstream of the POD to Rosamond Dry Lake is not known.

6 REFERENCES

Antelope Valley Technical Committee. June 26, 2008. Problem Statement Report: Antelope Valley Area of Adjudication.

California Geologic Survey. 2005. Seismic Hazard Map: Del Sur Quadrangle.

California Department of Water Resources. 1966. Water Wells in the Eastern Part of the Antelope Valley Area, Los Angeles County, CA, Bull. 91-12, 1 Plate (prepared by USGS: Koehler, J.H.).

California Department of Water Resources. 1962. Data on Water Wells in the Edwards Air Force Base Area, CA, Bull. 91-6, 208p., 1 Plate (prepared by USGS: Dutcher, L.W., et.al.).

California Geologic Survey (CGS). 2002. Seismic hazard evaluation of the Ritter Ridge 7.5-minute quadrangle, Los Angeles County, California

Christensen, A.H. 2004. Generalized water-level and long term water-level changes, September-October 2000 and March-April 2001, at the U.S Airforce Plant 42 and vicinity, Palmdale, California.

CGS. 2005. Seismic hazard evaluation of the Lancaster West 7.5-minute quadrangle, Los Angeles County, California

CGS. 2005. Seismic hazard evaluation of the Lancaster East 7.5-minute quadrangle, Los Angeles County, California

CGS. 2005. Seismic hazard evaluation of the Rosamond 7.5-minute quadrangle, Los Angeles County, California

City of Lancaster. 2005. Engineer's Report relative to the Revised Master Plan of Drainage

Corbett, Edward S., and Crouse, Robert P. 1968. Rainfall interception by annual grass and chaparral losses compared. Berkeley, Calif., Pacific SW. Forest & Range Exp. Sta. 12 pp., illus. (U.S. Forest Serv. Res. Paper PSW-48)

Dingman, Lawrence S. 2002. Physical Hydrology: Second Edition. Prentice Hall, New Jersey.

Durbin, T.J. 1978, Calibration of a mathematical model of the Antelope Valley ground-water basin, California: U.S. Geological Survey Water-Supply Paper 2046, 51.

Fetter, C.W. 2001. *Applied Hydrogeology: Fourth Edition*. Prentice Hall, New Jersey.

French, Richard H., Miller, Julianne J., Dettling, Charles, and Carr, James. R. 2006. Use of remotely sensed data to estimate the flow of water to a playa lake. Journal of Hydrology 325 (2006) 67-81.

Izbicki, John A., Flint, Alan L. and Stamos, Christina L. 2007. Artificial Recharge through a Thick, Heterogeneous Unsaturated Zone. GROUND WATER. Vol. 46, No. 3—May-June 2008 (pages 475–488).

Jennings, C.W. and Strand, R.G. 1969. Geologic Map of California: Los Angeles Sheet

Johnson, A.I., 1967. Specific yield-compilation of specific yields for various materials. *US Geol. Surv. WaterSupply Pap., 1662-D*, p. 74.

Keith Companies- North Counties, Inc. 1995. Hydrology and Hydraulic Design Storm Drain Facilities: Proposed Detention Basin B on Amargosa Creek.

Kennedy and Jenks. 2007. Antelope Valley Integrated Regional Water Management Plan

Leopold, L. B., Wolman, M. G., and Miller, J. P., 1964. Fluvial process in geomorphology: W. H. Freeman and Company, San Francisco.

Los Angeles County Department of Public Works (LACDPW). January 2006. *Hydrology Manual*.

LACDPW. April 9, 2008. Personal Communication. Rainfall Data Request.

Metzger, Loren F., Ikehara, Marti E., and Howle, James F. 2002. Vertical-Deformation, Water-Level, Microgravity, Geodetic, Water-Chemistry, and Flow-Rate Data Collected During Injection, Storage, and Recovery Tests at Lancaster, Antelope Valley, California, September 1995 through September 1998 . US Geological Survey Open File Report 01-414.

National Climate Data Center (NCDC). 2008. http://www.ncdc.noaa.gov/

NCEP/EMC . U.S. Gridded Multi-Sensor Rainfall (4 km) [NCAR/EOL] URL: http://data.eol.ucar.edu/codiac/dss/id=21.089.

Pacific Advanced Civil Engineering Inc. (PACE 2003a). *Amargosa Creek Double Barrel Arched Culvert Improvements, Hydrologic & Hydraulic Design Report*. February 2003.

PACE. 2003b. Amargosa Creek Soil Cement Channel Improvements. April 2003, revised August.

PACE. 2003c. Bayshore L.L.C. Amargosa Creek Soil Cement Channel Improvements (South Side). June.

PACE. 2004. *Amargosa Dam Maintenance and Operation, Dam No. 789*. May 2004

Ponti, D.J., Burke, D.B. and Hedel, C.W., 1981, Map showing Quaternary geology of the central Antelope Valley and vicinity, California: U.S. Geological Survey Open-File \Report 81-737, scale 1:62,500. *(Note: Quaternary geology layer was taken from CA Seismic Hazards evaluations)*

Regional Water Management Group. 2007, Antelope Valley Integrated Regional Water Management Plan. Prepared with assistance from Kennedy-Jenks.

Science Applications International Corporation (SAIC).. Technical Memorandum: *Amargosa Creek Percolation Demonstration*. July 2007.

SAIC. June 19, 2008.Technical Memorandum “City of Palmdale, Amargosa Creek, Water Right Application, Engineering Analysis”

State Water Resource Control Board (SWRCB).. *Example Format for WAA/CFII Report*. April 2006

United States Army Corps of Engineers. April 2008. Hydrologic Modeling System HEC-HMS User's Manual.

United States Army Corps of Engineers. December 2003. Geospatial Hydrologic Modeling Extension HEC-GeoHMS User's Manual.

USGS, 2003. Simulation of Ground-water Flow and Land Subsidence, Antelope Valley Ground-Water Basin, California

United States Geologic Survey (USGS). 2008. National Elevation Dataset: http://seamless.usgs.gov Dewberry. 2006. Vertical Accuracy Assessment Report 2006 LiDAR Bare-Earth Dataset for Los Angeles Region – Imagery Acquisition Consortium (LAR-IAC). October 26.

WEST Consultants, Inc (WEST). 2006. Hydraulic Analysis and Scour Evaluation for the 20th Street Bridge Over Amargosa Creek, City of Palmdale, CA.